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PAPERS, REPORTS, DISCUSSIONS, AND MEMOIRS

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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BALDWIN FILTRATION PLANT, CLEVELAND, OHIO

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Synopsis

In providing for the growth of American cities, one of the major problems is that of maintaining an adequate and sanitary water supply. The rapid increase in population in Cleveland, Ohio, during the past two decades has necessitated a careful study of this particular problem. For more than fifteen years, the Engineering Division of the Department of Public Utilities has investigated, planned, and constructed means for providing an ample supply of pure water.

While the location of Cleveland on Lake Erie permits of an ample supply of water for the city proper as well as for a Metropolitan District involving about 500 sq. miles, the need for providing water of the proper quality had not received much consideration until about 1913. The City of Cleveland supplies water to about thirty-five political subdivisions of Cuyahoga County and in several communities outside that county. For the most part, these do not border upon the lake and, therefore, it has been necessary to consider the problem of water supply for Cleveland as one that in reality involved a Metropolitan District.

The object of this paper is to describe a part of the development of a comprehensive plan for supplying this district with pure water. Although the city had constructed a filtration plant on the west side of the Cuyahoga River, which went into operation in 1918, only about one-half the supply was filtered until the Baldwin Plant was placed in operation in 1925. The water drawn through the city's intakes in Lake Erie is not as grossly polluted as many of the sources of public water supply of larger cities, but the growing

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population adjacent to the lake in this vicinity and the increased discharge of sewage directly into the lake have produced conditions that demanded an efficient purification system for the public water supply. This paper describes a large filtration plant into which has been introduced the most modern methods in the art of water purification and the largest covered concrete reservoir in the world. The paper has been subdivided as follows: History and Development of the Project; General Description of Flow of Water; Hydraulic Data; Design; Construction; General Equipment; Completion of Baldwin Filtration Plant; and Amount of Contracts and Analysis of Costs.

History and Development of the Project

The southern portion of Lake Erie opposite the City of Cleveland is the source of supply for all water furnished by the municipality to the city proper, its suburbs and neighboring villages, as well as to the more distant parts of Cuyahoga County and a small part of Lake County to the east. Because this water is low in turbidity, averaging about 15 parts per million, no attempt was made until late in 1911 to treat the water furnished for consumption, in any way, except by chlorination. At that time, after a more or less prolonged controversy regarding the necessity of filtering the water supply, a decision was reached to start plans for a filtration plant. About the beginning of 1918, the Division Filtration Plant, on Division Avenue, between West 29th and West 53d Streets, with a capacity of 150 000 000 gal. per day, was placed in service, thus giving a section of the City of Cleveland its first supply of filtered water. In general, this plant was capable of supplying only that part of the city and vicinity lying west of about East 55th Street. The remainder of the area supplied continued to be furnished with chlorinated raw water only. However, before the Division Filtration Plant was completed, plans were under way for a new plant which, in conjunction with that at Division Avenue, would be capable of supplying the entire city and its principal suburbs with filtered water.

In line with the usual procedure of locating a filtration plant near the pumping station supplying it with raw water, the original plans for this plant contemplated its construction near the Kirtland Pumping Station, on the lake front, at the foot of East 49th Street. A study of this location, however, indicated that conditions were such as to require pile foundations for all structures. The area available on which a plant could be built was insufficient without a considerable fill being made in the lake, and this, in turn, necessitated the use of a more or less expensive coffer-dam type of construction. Because of the difficulties to be overcome at the lake front site, the Engineers of the Cleveland Water Department gave consideration to the possibility of utilizing some other location.

It became evident about 1907, that the existing Fairmount Reservoir, supplying the low-service district, was becoming inadequate for storage purposes. It was also desired to increase the pressure in this service. Accordingly, during 1908 and 1909, the City purchased land for a new low-service distributing reservoir. This land was located directly east of Fairmount

Reservoir and extended from Baldwin Road to East Boulevard. The new land was about 60 ft. higher than the surface of the water in the old reservoir, making possible increased pressures in the area to be served.

A study of the possibility of utilizing the area in the vicinity of Baldwin Reservoir as another site for the new filtration plant was given serious thought. An estimate of cost revealed the fact that, with all factors taken into consideration, the Baldwin site was the cheaper. This finding was concurred in by an independent commission of engineers, composed of Messrs. J. W. Frazier, J. H. Herron, and Robert Hoffman, Members, Am. Soc. C. E., in a report dated June 1, 1920. As a result, the Baldwin Filtration Plant with a capacity of 165 000 000 gal. per day, was located on the Baldwin site, along the north, east, and south sides of Baldwin Reservoir. This location is approximately 3½ miles from Lake Erie.

The area within which the Baldwin Filtration Plant is located is approximately 50 acres. The filtration plant structures occupy approximately 11 acres and the Baldwin Reservoir about 13 acres. Fig. 1 shows the general location of the plant and the direction of flow of water.

The territory which the City of Cleveland will ultimately supply with water is a large metropolitan district destined to cover the entire area of Cuyahoga County and even to spread beyond its borders. The two existing filtration plants, having a total capacity of 315 000 000 gal. per day will supply all needs until about 1930. By that time a third plant will be required in the eastern half of the county, and about ten years later a fourth in the western half.

General Description of Flow of Water

The raw water for the Baldwin Filtration Plant is drawn from Lake Erie through a steel and concrete crib about 4 miles offshore, and thence through a 9-ft. brick-lined tunnel about 26 000 ft. long to the Kirtland Pumping Station. From there it is pumped through one lock-bar steel and three cast-iron raw-water mains, each 48 in. in diameter and about 7 500 ft. long. These four mains combine into two 60-in. lock-bar steel pipe lines, each approximately 15 000 ft. long, leading to Fairmount Reservoir. The water is then lifted by centrifugal pumps in Fairmount Pumping Station (see Fig. 1) and is forced through two 60-in. cast-iron mains, each approximately 2 100 ft. long, up the rising well in the chemical house of the filtration plant, to the upper pool of the three hydraulic-jump mixing flumes (capacity of each flume, 55 000 000 gal. per day).

Passing through these flumes (Fig. 1), the water flows through a channel approximately 124 ft. long, containing two pairs of under and over-baffles; and thence through Gate-House No. 1, into the south conduit of the coagulating basins. Here it is distributed through two gate-houses into the four basins themselves. The water enters the coagulation basins at the south end over submerged weirs, travels through them, and leaves at the north end over similar weirs. The water then flows through the control gates and a conduit to the Administration Building, passing on its way through Gate-House No. 6.

The flow of water divides in the Administration Building, supplying the filters as shown in Fig. 1. From the filters, it passes through rate controllers into effluent galleries or small storage reservoirs. There is one gallery under each group of ten filters. The water flows from each of the galleries to a central conduit in the basement of the Administration Building, whence it divides and passes through sluice-gates into either one or both of the two basins of Baldwin Reservoir.

Water is delivered to these basins by means of multiple-weir flumes along the north side. It then passes across the basins and is collected on the south side at the bottom by means of openings in the conduits leading to Gate-House No. 7. There, the water is distributed by two 36-in. and three 48-in. mains into the low-service district of the city. Two other 48-in. mains leading from Gate-House No. 7, act as the suction mains for the first and second high-service pumps in Fairmount Pumping Station. The first high-service district is supplied from this station through two 30-in. mains and one 24-in. main; and the second high-service district, through one 30-in. main and one 48-in. main.

Hydraulic Data

In the raw-water mains under full plant capacity, the computed velocity in the 48-in. mains is 3.3 ft. per sec., and in the 60-in. mains, 4.2 ft. per sec. With normal operating levels in the coagulation basins, the computed velocity in the channel leading from the hydraulic-jump mixing flumes to Gate-House No. 1, varies from 0.76 ft. to 1.20 ft. per sec., except at the under and over-baffles where a velocity of 2.53 ft. per sec. is reached. In the influent conduit of the coagulation basins, the computed velocity of the water is 2.17 ft. per sec. and in the effluent conduit, 2.42 ft. per sec., the latter being also approximately the maximum velocity through any of the sluice-gates. The flow through the coagulation basins is at a computed rate of 2.36 ft. per min. The computed velocity through the settled-water conduits is 2.37 ft. per sec., and through the settled-water piping leading to the filters, 2.04 ft. per sec.

It will be noted that the computed velocities of flow through conduits, sluice-gates, etc., after the water has passed through the hydraulic jumps, is less than 2½ ft. per sec. This is the value above which it was not desired to go on account of the possibility of breaking up the floc; but this velocity is exceeded if the coagulation basins are by-passed.

The computed loss of head through the filter plant, together with the actual losses obtained as the result of a series of observations made when the plant was operating at the nominal rated capacity of 165 000 000 gal. per day with water at normal operating level in the coagulation basin, is shown in Table 1. The entire filter plant was designed so that it could be operated with the hydraulic gradient 1 ft. above normal levels in order to take care of operating fluctuations.

Design

The following data and unit stresses were used in designing the various parts of the work, except as otherwise noted:

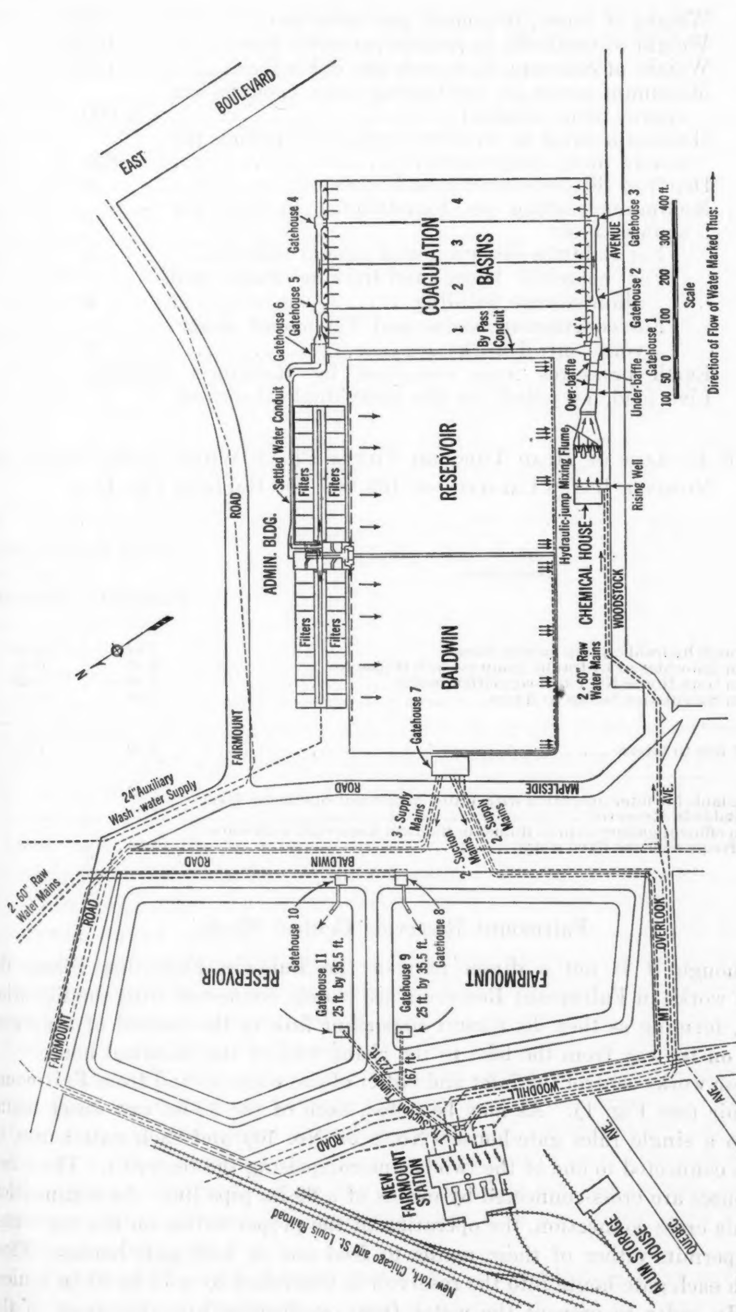

FIG. 1.—GENERAL ARRANGEMENT OF STRUCTURES AND DIRECTION OF FLOW OF WATER.

Weight of water, in pounds per cubic foot	62.5
Weight of earth-fill, in pounds per cubic foot	100.0
Weight of concrete, in pounds per cubic foot	150.0
Maximum stress in reinforcing steel (pounds per square inch, tension)	16 000
Maximum stress in reinforced concrete (pounds per square inch, compression)	650
Depth of fill over structures, in feet	2
Maximum bearing on foundations, in tons per square foot:	
For Baldwin Reservoir and effluent galleries	5
For chemical house and mixing flume and alum storage building	3
For coagulation basins and Fairmount Reservoir control works	2

Earth pressures were computed by Rankine's formula.
Live load, as noted for the individual structures.

TABLE 1.—Loss of Head Through Filter Plant When Operating at the Nominal Rated Capacity of 165 000 000 Gallons Per Day.

Description.	Loss of Head, in Feet.	
	Computed.	Observed.
Loss through hydraulic-jump mixing flumes	2.04	2.40
Loss from tail-water of hydraulic jump to Gate-House No. 1	0.97	0.88
Loss from Gate-House No. 1 to coagulation basins	0.56	0.30
Loss from coagulation basins to filters	1.25	0.93
Total loss to filters	4.82	4.51
Head available for filter operation with water at normal operating level in the Baldwin Reservoir	11.75	....
Loss from effluent gallery to inlet flumes in Baldwin Reservoir, with water falling freely over the fixed weirs	1.00	0.81

Fairmount Reservoir Control Works

Although it is not a direct part of the Baldwin Filtration Plant, the control works in Fairmount Reservoir are closely connected with the filtration system, forming as they do, a most important link in the control of the water supply on its way from the lake to the rising well of the chemical house.

These works control the inlet and outlet of the water to and from Fairmount Reservoir (see Fig. 1). As may be noted, each of the 60-in. raw-water mains leads to a single inlet gate-house (No. 8 or No. 10) and each gate-house, in turn, is connected to one of the two basins comprising the reservoir. These two gate-houses are cross-connected by means of a 60-in. pipe line. In conjunction with this cross connection, the operation of the proper valves on the raw-water mains permits either of these mains to feed one or both gate-houses. Flow through each gate-house into the reservoir is controlled by a 72 by 84-in. sluice-gate. In order to prevent the water from overflowing into the street, if the sluice-gates in both gate-houses are improperly closed, each gate-house is

provided with an overflow, which is directly connected to the Baldwin Road sewer, and is capable of taking the flow of one-half of the filter plant capacity.

From each gate-house, a 72-in., riveted steel pipe passes through the reservoir embankment and down the slope to the floor of the reservoir. In the section through the embankment the pipe is entirely surrounded by concrete. A cut-off wall is also provided which is connected to a concrete apron on the slope to prevent water working its way to the outside of the reservoir. The remainder of the pipe on the slope and reservoir floor rests in a concrete cradle to which it is anchored. Due to the possibility of damage by ice to the part of the pipe on the slope, provision is made near the top and bottom, by means of flanged joints, for replacing this part of the inlet pipe. The section of the pipe laying on the reservoir floor slopes upward somewhat and increases in size from 72 to 96 in. in diameter. It is laid so as to point approximately to the diagonally opposite corner of the reservoir. These three points of design—the larger end, slowing up the velocity; the upward trend, tending to widen vertical distribution; and the angle, directing the water toward the opposite corner from that in which the outlet gate-house is located—are incorporated so as to assist in a somewhat better distribution of water in the basins.

Each of the outlet gate-house structures (Nos. 9 and 11, Fig. 1) has three water passageways, 4 ft. 8 in. in width, connecting the reservoir to a central well 12 ft. in diameter. Each passageway is controlled by a 42 by 72-in. sluice-gate at the central well. To permit water to be drawn from the reservoir at different levels, a separate port, opening into each passageway was left in the outside wall of the gate-house. One of these was at the bottom of the reservoir, one at the top, and the third midway between the top and the bottom. Screens and stop-planks are provided in each of the three passageways. The walls of the well and passageways support an operating floor approximately 5½ ft. above high water in the reservoir. The bottom of the well is connected, by means of an inverted truncated cone, to a shaft 7 ft. in diameter. This latter leads to a tunnel 7 ft. in diameter, the center line of which is approximately 28 ft. below the floor of the reservoir.

The foundation of each gate-house has approximately 6 ft. 6 in. of rubble concrete topped with 2 ft. of plain concrete. The bottom of this foundation rests on hard shale and the top is just a little above the reservoir floor. The three passageways and central well are formed by heavily reinforced concrete sections. As a protection against ice in the reservoir, all exterior concrete is faced with granite to a point approximately 5 ft. below the overflow.

The tunnels (Fig. 1) leading from the outlet gate-house shafts converge and form one tunnel of the same diameter at a wye-intersection. Continuing from this wye, the tunnel extends a distance of approximately 129 ft., where it joins a 7-ft. circular conduit, leading to the header on the suction of the low-lift pumps in Fairmount Pumping Station. The junction between each of the two vertical shafts and the tunnels is made with long-sweep radii in order to reduce loss of head.

The tunnel is bored through shale. Its cross-section is, in general, circular with a minimum thickness of shell of 12 in. At the approach to the wye,

the section is made elliptical, 5 ft. 6 in. by 9 ft., with the long diameter vertical, in order to reduce to a minimum the width of unsupported roof at the wye proper. Except at the wye, where the roof becomes flat, the tunnel is composed of plain concrete. At the wye, ⅜-in. square steel reinforcing hoops were placed on 4-in. centers. Provisions were also made in the shafts and tunnel for bulkheading in case it were necessary to use compressed air.

Chemical House and Mixing Flumes

General Description.—The arrangement of the chemical house, mixing flumes, and channel is shown in Fig. 2. With the exception of the upper story of the chemical house, all the structures are of reinforced concrete of the beam-and-slab type with independent column footings. The upper story consists of brick bearing walls with steel roof trusses. The remainder of the building proper is of the skeleton type with brick curtain-walls. Basic data concerning this unit (see Fig. 2) may be listed, as follows:

Nominal capacity of the plant, in million gallons daily	165
Elevation of water surface in the rising well	245.74
Concerning a point in the channel just before the hydraulic jump:	
Depth of water, in feet	0.47
Width of three flumes, in feet (each flume)	14.56
Velocity of water, in feet per second	12.5
Elevation of water surface	242.82
Concerning a point at the foot of the slope in the mixing chambers; nominal operating conditions:	
Depth of water, in feet	3.19
Width of water passages, in feet (three each at 22 ft.)	66
Velocity of water, in feet per second	1.13
Elevation of water surface	243.69
Loss of head, in feet, between the rising well and the foot of slope in the mixing chamber	2.05
Loss of head, in feet, between the foot of slope in the mixing chambers and the coagulation basins	1.69
Distance, in feet, down stream from the top of slope in the mixing chambers, to the location of the hydraulic jump	8
Flow-line elevation in the coagulation basins	242.00
Alum Storage Bins:	
Gross capacity, in tons	450
Net capacity, in tons	350
Dissolving Tanks:	
Volume, in cubic feet per tank, to the top of overflow plank weir	157
Charge of alum required, in pounds per tank	4 000
Solution Tanks:	
Capacity of each tank, in gallons	9 600
Alum, in 5% solution, required for each tank (tons)	2
Depth, in feet, of the solution in the tanks	4.62
Summary of Alum Required:	
Maximum volume of water treated, in million gallons daily	165
Alum, in grains per gallon	1
Rate, in tons per day, in which alum is applied	11.8
Speed of bucket hoist, in feet per minute (capacity, 3 000 lb.)	20
Speed of passenger elevator, in feet per minute (capacity, 2 500 lb.)	150

Alum storage capacity, in tons, required to treat 165 000 000 gal. daily with 1 grain per gallon:	
Baldwin Chemical House (30-day supply)	350
Alum Storage Building (85-day supply)	1 000
Baldwin Chemical House, on the dissolving-tank floor (20-day supply)	225
Total storage capacity (135-day supply), approximately	1 575
Height, in feet, to which bags of chemicals may be stored	3

Chemical Bin Structures.—The chemical bins (Fig. 2) occupy the major portion of the structure to the west of the rising well. They consist of two separate bins with a passageway between them, and with the top of the bins situated at the general ground level. This total net capacity is about 350 tons. The westerly bin extends the full width of the building, and the easterly one, a little less than one-half this distance. They are continuous, with seven hoppers in the large bin and three in the small one. The minimum side slope is approximately 45 degrees. The bins discharge through 12 by 14-in. steel chutes equipped with single undercut gates, operated with a chain-and-link mechanism. These chutes discharge into a bucket on a small industrial or transfer car that is spotted in a common passageway beneath the bins. The cover over the alum bins forms the floor of the receiving room (see Section *D-D,* Fig. 2). In this floor there is one manhole to approximately 90 sq. ft. of horizontal bin surface.

Chemical Tanks and Piping.—On the upper floor of the chemical house are located the tops of six dissolving and four solution tanks. These are divided evenly into two independent groups, each of which is connected with a double orifice box unit. The arrangement is such that the central dissolving tank in each group is capable of feeding either solution tank. A system of piping conveys the solution from the solution tanks to the orifice boxes and from there to the lead-lined distributing trough immediately over the rising well. Piping is also provided to convey the solution to a point of secondary application in Gate-House No. 6 which is in the outlet conduit from the coagulation basins.

All chemical piping is flanged cast iron and all valves are of acid bronze. In all cases in which this piping is joined to chemical tanks, special wall castings, no parts of which come in contact with the chemical solution, were provided. This will permit complete replacement of the chemical lines at any future time and at a minimum expenditure.

Hydraulic-Jump Section.—The raw water is introduced to the mixing-flume section, at its westerly end (see Fig. 2), by means of a rising well which extends entirely across the width of the chemical house. The raw water enters the rising well from two 60-in. mains.

Flowing from the upper pool, the water passes through three hydraulic-jump mixing flumes. These flumes have expanding sides and sloping bottoms, and are preceded by horizontal throats of constant width with curved entrances. At the foot of the slopes, they lead into a converging horizontal flume (see Fig. 2). Provisions have been made in each throat for a double row of stop-planks in order to cut off the flow of water through the flume, if desired. The flumes are housed within a one-story building; walk-ways, conveniently located, permit an adequate view of the hydraulic jumps.

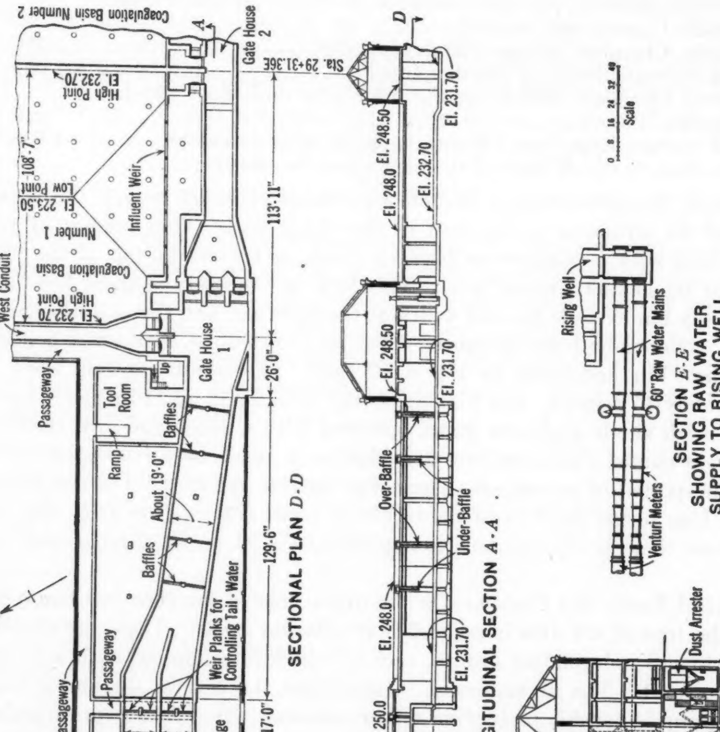

FIG. 2.—CHEMICAL HOUSE: GENERAL PLAN AND SECTIONS.

Just beyond the weir planks for controlling the tail-water depth, the converging shallow flume is transformed to a conduit section (see Fig. 2). This is accomplished by dropping the floor about 9 ft. and narrowing the sides, thus producing a conduit section about 19 ft. wide and 13 ft. deep leading to Gate-House No. 1. This conduit section, which is about 90 ft. long, has a pair of under and over-baffles near each end.

In order to present a more pleasing external appearance, the exterior walls of the chemical house were extended through to Gate-House No. 1, and the area between these walls and the conduit was roofed over at the level of the top of the conduit. Over this entire area an earth-fill, 2 ft. deep, was provided.

Parts of the space between the conduit and the exterior walls were utilized for garage, tool-room, and covered passageway. This passageway is part of a complete system, leading from the chemical house, along the coagulation basins, to the filter building. Beneath it is a tunnel in which are located the steam and water lines, secondary chemical feed pipes, and conduits for light, power, and telephone cables.

Basis of Design.—Except in the case of water passages, where maximum water heights are used, the usual live load is 75 lb. per sq. ft. On the floor where the dissolving tanks are located—owing to the possibility of storing sack alum—a live load of 250 lb. per sq. ft. is used. The floor over the alum storage bins is designed to carry a motor truck which, when loaded, weighs about 7 tons.

Expansion joints, extending down through the foundation, are provided at six places in the chemical house and mixing flumes. They are placed where pronounced changes are made in the general type of the structure, and, in addition, at one or two other places, so that in no case is there a distance between joints greater than 82 ft. The shape and proportions of the hydraulic-jump flumes, as well as the dissolving and solution* tanks, are based on experiments conducted by the Engineers of the Cleveland Water Department.†

Coagulation Basins

The coagulation basins shown in Fig. 1, extend for a distance of nearly 500 ft. from the east wall of Baldwin Reservoir. Data pertaining to these basins are, as follows:

Over-all area, in square feet, including conduits and gate-houses (457 ft. by 747 ft.)	341 379
Area, in square feet, between weirs inside of basins (108 ft. 7 in. by 661 ft. 6 in.):	
Basin No. 1	70 500
Basins Nos. 2, 3, and 4 (each)	70 900
Depth of water, in feet:	
Maximum	18.5
Minimum	9.3
Average	15.0

* "Rate of Solution of Sulphate of Alumina," by J. W. Ellms, A. G. Levy, and L. A. Marshall, *Journal*, Am. Water Works Assoc., July, 1921.

† "The Hydraulic Jump as a Mixing Device," by A. G. Levy and J. W. Ellms, *Journal*, Am. Water Works Assoc., January, 1927.

Capacity, in gallons, each basin	8 200 000
Detention period between weirs, in hours	4.7
Velocity of flow through basin, in feet per minute	2.36
Velocity of water in south conduit, in feet per second (water surface elevation, 242.5)	2.17
Velocity of water in north conduit, in feet per second (water surface elevation, 241.5)	2.42
Maximum velocity of water through sluice-gates (in feet per second)	2.43
Effluent and influent weirs:	
Maximum elevation	240.5
Minimum elevation	237.0
Length of weirs in each basin, in feet	77.0
Stilling-baffle:	
Elevation of top	242.5
Elevation of bottom	237.71
Overflow weirs:	
Length, in feet (each basin)	28.27
Depth, in feet, above weir to discharge 41 250 000 gal. daily (each basin)	0.77
Elevation of top	243.0
Elevation of floor:	
At center line	223.5
At sides	232.7
Elevation of flow line	242.0
Elevation of intrados at crown	245.5
Distance, in feet, from floor to crown of arch at the center of the basin	22.0
Total number of 20-in. square columns in the four basins, 15 ft. 9 in. on centers	1 041

The four basins have an average water depth of 14 ft. 11 in., varying from a minimum of 9 ft. 4 in. to a maximum of 16 ft. 6 in. The water enters at one end of the basin over a weir and under a stilling-baffle and is taken off at the opposite end over a similar weir. Provisions have been made for by-passing the coagulation basins through a conduit along the west side of the basins, connecting Gate-Houses Nos. 1 and 6 (see Figs. 1 and 3). An inlet conduit forming the south wall of the basins furnishes the waterway through which the water flows to Gate-Houses Nos. 2 and 3. The former controls the flow to Basins 1 and 2, and the latter to Basins 3 and 4. There is one 60 by 84-in. sluice-gate for each basin. The control from the basins, at the outlet end, is effected through Gate-House No. 5 for Basins 1 and 2, and through Gate-House No. 4 for Basins 3 and 4. At the outlet end, there is one 60 by 84-in. sluice-gate for each basin.

The by-pass control of the coagulation basins is effected through Gate-Houses Nos. 1 and 6, where the water may be shut off entirely from the north and south conduits. When the by-pass is in use all the water flows through two 60 by 84-in. sluice-gates in each of the gate-houses. Under the condition of maximum flow, a velocity through these gates of about 3¾ ft. per sec. results.

Each basin is of reinforced concrete construction with groined arch roof, reinforced sloping floor, and independent column footings. (See Fig. 4.) The floor consists of two parts, a sub-floor 4 in. thick and a finished floor 8 in. thick, the latter being reinforced with ½-in. square deformed bars on 15-in.

centers in both directions. The sub-floor butts against the walls and column bases. The finished floor rests on a ledge at the walls and at the column footings, and all floor joints are filled with asphalt to prevent leakage. The sloping floor performs the dual function of facilitating cleaning and of reducing the height of the wall sections.

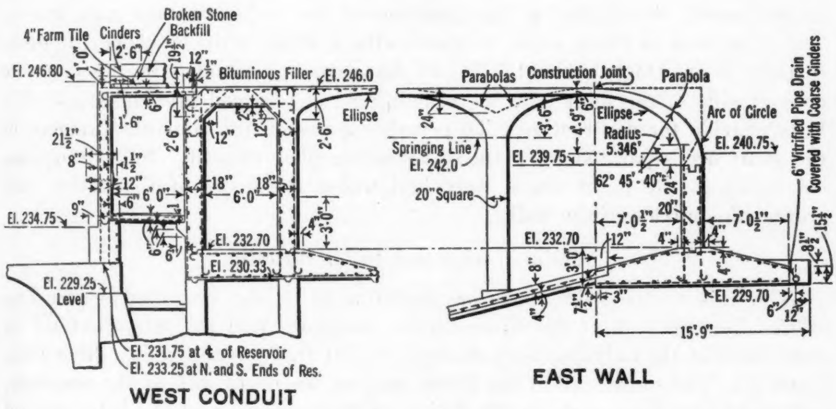

FIG. 3.—COAGULATION BASINS: DETAILS OF WEST CONDUIT AND EAST WALL.

All conduits (Fig. 3) were designed as rigid structures. Stability was investigated under conditions allowing an upward hydrostatic head of one-third the height of the water in the basin. In the case of the north conduit, recognition was taken of the possible removal of the earth backing at some future time to take care of adjacent construction, and the conduit was designed accordingly. At the by-pass conduit, the usual barrel arch was designed as a cantilever, extending from the roof of the conduit. Because of the frequency with which it has been observed that arch action does not always take place between the barrel and groined arches, all dividing walls between the basins were designed on the basis of an unbalanced arch thrust. The groined arches are tied together in groups of four and reinforced with two ½-in. square bars on either side of the crown. All manholes are tied securely to the groined arch roof with steel reinforcement extending well into the arches. This is required particularly in the case of the manholes supporting the mechanism for the mud-valves, in order to prevent frost action between the roof and manholes lifting the mud-valves off their seats in the floor of the basin, and thus permitting water to leak into the drains. The columns supporting the roof are of five different lengths, varying from 10 ft. 8 in. to 17 ft. 11 in. In the 10 ft. 8-in. and 12 ft. 8-in. columns, no reinforcement is used. The remaining columns are reinforced with four ¾-in. square bars, with ¼-in. round ties spaced 12 in. center to center. Six stiffening walls are placed in each basin, parallel to the long axis, and near the center of the basin.

Each coagulation basin is provided with six mud-valves for cleaning and draining. These valves permit flow into a lateral drain placed immediately

under the floor of the basin and along the longitudinal axis. Each drain is connected into the main drain header near the north end of the basin.

To assist in cleaning, 2½-in. hose gates spaced about 50 ft. from center to center are provided on each side of each basin. They are connected to 6-in. water mains placed on the long walls of the basins.

Expansion joints are placed in the coagulation basin walls at intervals of approximately 60 ft., and at the junction of the walls with the gate-houses. The water-seal at these joints is made with a 16-in. strip of copper, approximately $\frac{1}{16}$ in. thick. About 7 in. of the strip is embedded in the concrete on each side of the joint with the remaining 2 in. formed into the shape of a V to provide for contraction. To permit expansion, the concrete surfaces at the joint are separated by $\frac{3}{8}$ in. of asphaltic fiber cement. Subsoil drains, connected to the main drain, were laid under the floors of the basins and around the outside of the walls.

Administration and Filter Building

The axis of the Administration Building is on the projected center line of the dividing wall of the filtered-water reservoir, and the filters extend on either side of the building for a distance of 391 ft. from this axis. (See Figs. 1 and 5.) The south wall of the filters rests on the north wall of the reservoir, and provisions were made in the design so that a portion of the inlet control works of this reservoir forms the foundation for a part of the Administration Building.

Administration Building.—The Administration Building is a skeleton steel structure 166 by 71 ft. in plan and, approximately, 51 ft. in height from the main floor to the under side of the roof trusses. Below the main floor (Elevation 244) are a basement and an intermediate floor. The general offices are on the main floor, with the store rooms, transformer room, pipe shop, blacksmith shop, and chlorine room on the floor below (Elevation 234). The main piping, carrying the raw and filtered water, is on the lowest or basement floor. The floor containing the laboratories, general storeroom, and machine shop, and the drip floor, located just below the wash-water tanks, are above the main floor. Access is furnished to all these floors by stairs and a freight elevator.

The architectural arrangement is such that an elaborate stone double stairway leads to the main entrance to the building (Fig. 5), and advantage was taken of the space under this stairway for locating the wash-water pump-room. This room is 21 ft. 6 in. by 51 ft. 6 in. in plan, and access to it is by means of a tunnel from the basement of the Administration Building.

Settled Water Conduit.—Water is conveyed from the coagulation basins through a concrete conduit, 8 ft. wide by 13 ft. 5 in. high, north of and adjacent to the east wing of the filter building. (See Fig. 1.) This conduit terminates in a section, 14 ft. 5 in. wide by 13 ft. 5 in. high, just within the Administration Building. The top of this section forms a part of the floor at Elevation 234.0. This latter conduit is connected to the conduit that runs the length of the pipe galleries, by a short section of double-barreled conduit beneath the floor at Elevation 234.0. Within the pipe gallery, the conduit is 7 ft. wide by 8 ft. high. In the Administration Building, it is 11 ft. 3 in. wide by 5 ft. high. The conduits within the pipe galleries are connected to

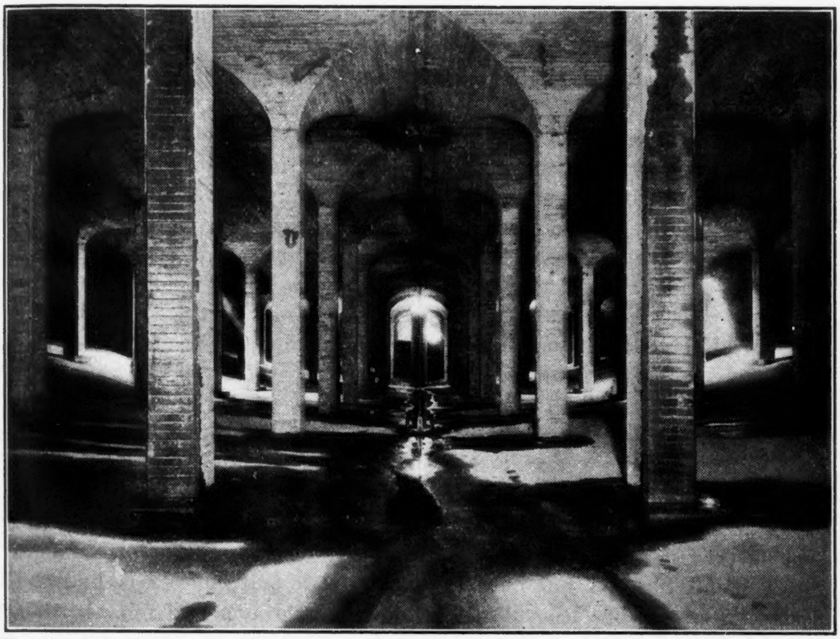

FIG. 4.—INTERIOR VIEW OF ONE OF THE COAGULATION BASINS, BALDWIN FILTRATION PLANT.

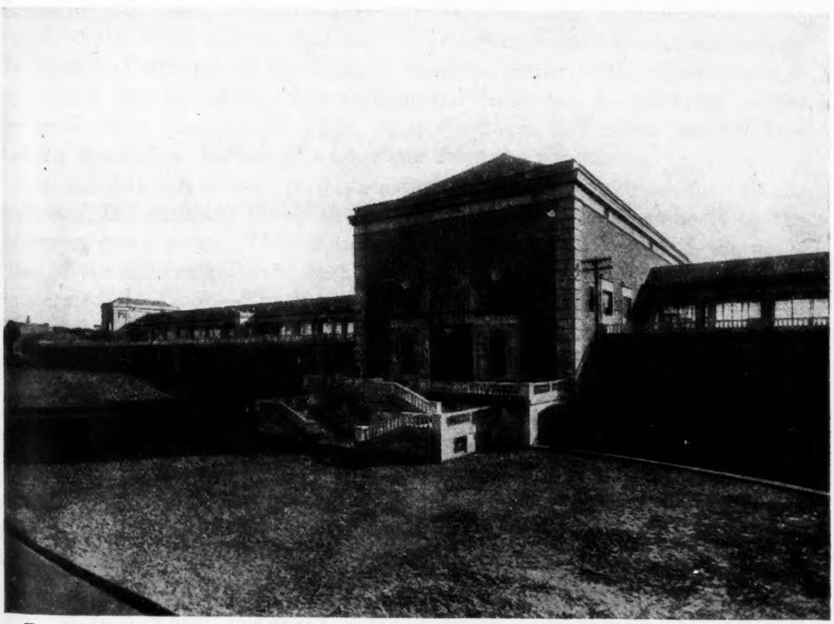

FIG. 5.—VIEW OF MAIN ENTRANCE TO ADMINISTRATION AND FILTER BUILDING, BALDWIN FILTRATION PLANT.

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that within the Administration Building by means of transition sections. The change in the conduit dimensions was brought about by the need of conserving head-room in the Administration Building and width of gallery in the pipe galleries themselves.

Filtered Water Conduit.—Along the north and south center line of the Administration Building and resting upon the basement floor, is a reinforced concrete conduit for conveying the filtered water from the effluent galleries to the filtered water reservoir. From the end of the conduit nearest the settled water conduit, a 54-in. by-pass is connected to the latter. This by-pass is controlled by two 48-in. valves, which are arranged with a spacer piece between them. A 2-in. open drain, in the bottom of the spacer-piece guards against the slightest possibility of contamination of the filtered water by the settled water. The filtered-water conduit is connected to the four filtered-water effluent galleries under the filters by 54-in. cast-iron pipe controlled by 48-in. valves. At the filtered-water reservoir end of the conduit, connection is made to the inlet control works of this reservoir. The control works contain four 60 by 84-in. sluice-gates, arranged in pairs, for controlling the flow of water to either or both of the two basins of the filtered-water reservoir.

Piping.—Under that part of the settled-water conduit, which runs along the east and west center line of the Administration and Filter Building, is a 30-in. wash-water line connected, by a cross located near the west side of the building, to a 42-in. line from the wash-water tanks and to the 24-in. discharge from the wash-water pumps. Four valves are grouped in the immediate vicinity of this cross, one on either side of it on the 30-in. line and one on each of the 24-in. and 42-in. lines. This arrangement permits wash-water to be furnished either from the tanks or from the pumps to the filters in either wing of the filter building. A man-hole and drain are also provided in the 30-in wash-water line. In the 24-in. pump discharge, a Venturi meter is provided for measuring the flow of wash-water from the pumps.

A 30-in. cast-iron suction pipe is encased in concrete, under the floor of the basement. It continues under the floor of the tunnel and connects to the wash-water pump-room. This suction pipe is connected to sumps in the floors of two of the effluent galleries, each branch being provided with a 30-in. valve.

Intermediate Floor.—The intermediate floor at Elevation 234.0 is located at the level of the top of the settled-water conduit. On this floor are rooms for storing and applying chlorine, a transformer room, and pipe and blacksmith shops. All deliveries to the Administration and Filter Buildings are made to this floor at a doorway beneath the main entrance. This door is reached by a semi-circular roadway passing under this entrance.

Main Floor.—On the main floor at Elevation 244.0, is the operators' room, in which are located various gauges. Adjoining this room is a dining room and kitchen for the use of employees. A drafting room, locker room, toilets, and space for a freight elevator complete the west half of this floor. Across from the operators' room in the southeast corner of the main floor is an observation room, in which is located, open to public view, one of the system of weirs which admit water to Baldwin Reservoir. Adjacent to this room is a

small assembly room. In the northeast corner of the building is a group of offices occupied by the superintendent and clerical forces.

Laboratory Floor.—On the laboratory floor at Elevation 258.67, are located the laboratories, machine shop, stock room, toilets, and locker room. The rooms used for chemical and bacteriological work include two chemical laboratories for water analysis, one for coal and oil testing, a preparation room for media, a bacteriological laboratory, a microscopic room, an incubator room, a refrigerator room, a photographic dark room, a library, and chemical storage rooms.

Wash-Water Tanks.—The two wash-water tanks are located on a steel grillage in the upper part of the Administration Building, with the bottom of the tanks at Elevation 281.12, or about 44 ft. above the maximum height of the wash-water troughs in the filters. These tanks are of steel plate construction, each 60 ft. in diameter and 12 ft. deep, and are intended to operate with a maximum of 10 ft. of water. At this depth the capacity of the two tanks is 423 000 gal., or approximately the quantity required for six filter washes. A 36-in. cast-iron pipe leads from the bottom of each tank into a header, which discharges into a 42-in., vertical, pipe line, leading to the basement where it joins the 30-in. wash-water line in the pipe galleries. The pipe from each tank is provided with a 36-in. valve for controlling the flow. A 16-in. cast-iron overflow is installed in each tank. The entrance to this overflow is over a bellmouth piece with its weir edge placed 10.25 ft. above the bottom of the tank. In order to reduce the tendency to suck air caused by vortex action under low-water conditions, an inverted steel cone 3 ft. 10 in. in diameter and with the sides forming an angle of 45° to the base, was placed at the entrance to each 36-in. outlet. The apex of the cone is set in the plane of the bottom of the tank. In order to facilitate the maintenance of the steel tanks and the grillage, as well as to take care of condensation from the tanks, pipe, etc., a reinforced concrete floor with a proper system of drainage is placed under the tanks at an elevation of approximately 272.0 ft.

Filter Building.—The filters are housed in two wings of the Administration and Filter Building. (See Fig. 5.) These wings are each approximately 360 ft. long by 130 ft. wide and extend at right angles to the major axis of the Administration Building. Statistics relating to these units are as follows:

Nominal capacity of filter beds, in million gallons per day......	165
Number of filter beds	40
Capacity, in million gallons per day per filter	4.16
Rate of filtration, in million gallons per day per acre	125
Clear distance between weirs of the wash-water troughs, in feet..	4.93
Capacity of wash-water troughs, in inches of rise per minute:	
With adjustable weir plates down.........................	24
With adjustable weir plates up...........................	33
Probable practical rates of rise, in inches of rise per minute:	
With adjustable weir plates down.........................	21
With adjustable weir plates up...........................	26
Wash-water capacity of main gutter, in inches of rise per minute.	33
Maximum velocity of flow through settled-water conduits, in feet per second	2.42

Maximum velocity of flow through 24-in. settled-water pipes, in feet per second	2.04
Elevations:	
Edge of wash-water troughs:	
Maximum	236.92
Fixed	236.50
Top of central gutter	237.50
Bottom of central gutter (sloped from Elevation 230.75)	230.50
Floor of effluent galleries	216.0
Floor of filters	230.5
Normal water-line in filters	240.63
Top of sand	234.83
Top of gravel	232.33
Floor of wash-water tanks	281.12
Overflow of wash-water tanks	291.37
Top of wash-water tanks	293.12
Filter sand:	
Depth, in inches	30
Effective size, range in millimeters	0.35 to 0.42
Uniformity coefficient	1.70
Filter gravel, depth of graded layers:	
Size, $1\frac{1}{2}$ in. to $\frac{3}{4}$ in	15
$\frac{3}{4}$ in. to $\frac{1}{4}$ in	3
$\frac{1}{4}$ in. to $\frac{1}{8}$ in	2
$\frac{1}{8}$ in. to $\frac{1}{16}$ in	2
Total depth of gravel, in inches	22
Capacity of wash-water pumps, in million gallons per day:	
Two at 1 850 gal. per min	5.33
Two at 3 450 gal. per min	9.94
Total capacity of four pumps	15.27
Capacity of two wash-water tanks (water, 10 ft. deep) each tank, in gallons	211,500
Diameter of wash-water tanks, in feet	60
Total weight of one full tank (10 ft. of water), in tons	946
Average quantity of water required to wash one filter, in gallons	71 000

Each group of ten filters is built upon the roof of an effluent gallery or small filtered-water reservoir into which these filters discharge. The filter operating floor is located at the level of the top of the filters and spans the pipe gallery.

Filter Tanks.—The filter tanks were designed to be built as monoliths. Each unit has a sand area of 1 450 sq. ft. divided into two equal areas by a center gutter. The outside walls of the tanks are designed as vertical slabs held at the bottom by the tank floor and at the top by the filter walks, acting as horizontal beams and as ties. The walks extend along the top of the four outside walls, over the central gutter, and at right angles to the center walk.

Wash-water gutters, running at right angles to the center gutter, are designed and arranged as shown in Fig. 6. Longitudinally, the slope of the bottom is 1 in. in the full length of the gutter.

The strainer system of the filter tanks is of the perforated-pipe manifold type, with the manifold located under the central axis of the sand bed in each half of the filter. The manifolds for each half filter consist of four independent castings, each having a central source of water supply. Each of the manifold castings supplies forty-two cast-iron laterals, arranged as shown in

Fig. 6. In order to wash the otherwise dead area over the manifold, ⅜-in. brass nozzles were provided, one nozzle being furnished for approximately 52 sq. in. of sand area over the manifold. The total area of the holes in the strainer system is 0.309% of the area of the sand bed. The area of the holes in one lateral is 36% of the cross-sectional area of the lateral, and the area of the laterals fed by one manifold is 71.4% of the cross-sectional area of the manifold. The effective length of the lateral is 29.5 times its diameter.

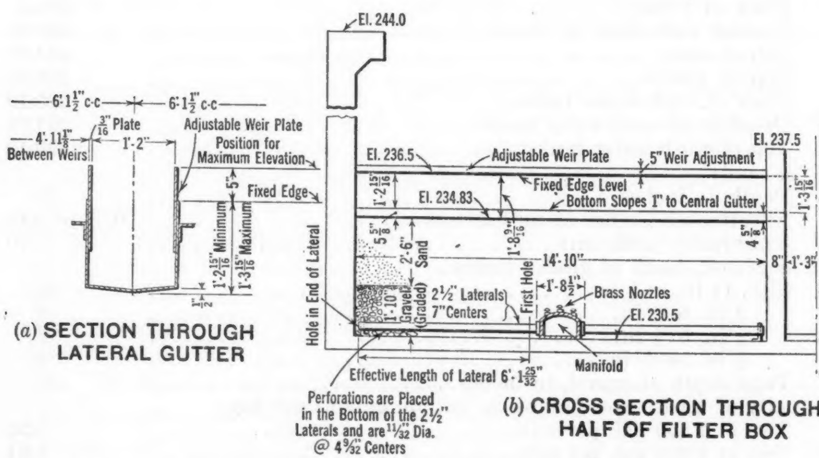

FIG. 6.—GENERAL CROSS-SECTION OF ONE-HALF OF A COMPLETED FILTER TANK.

The strainer system was designed in accordance with the results of experiments by one of the writers,* which were an extension of those made by Harry N. Jenks,† M. Am. Soc. C. E. The original design contemplated the use of flange connections for the laterals, but, on the basis of bids submitted, it was found that a considerable saving could be made by the use of the bell-and-spigot joint. To eliminate the possibility of slippage at the lead joint, a collar is cast on the lateral near the closed end. Cast-iron lateral chairs were designed to be slipped over the ends of the laterals as far as the collar and then to be bolted to the filter tank bottom. The manifolds are practically uniform in cross-sectional area. They are provided with hand holes near the end of each casting and with a 12-in. manhole at the center just over the inlet for the water supply.

Effluent Galleries.—The effluent galleries are four in number and each is 352 ft. 6 in. long. The two north galleries are each 47 ft. 6 in. wide and the south galleries, 45 ft. 3 in. wide. The distance from the floor to the under side of the roof is 13 ft. 3 in. The capacity of the four galleries is about 6 000 000 gal. Each gallery is provided with two 12-in. drains. These drains empty into the main drain and each is provided with two valves in series. A 24-in.

* "Design of Perforated Pipe Strainer System," by J. W. Ellms, M. Am. Soc. C. E., *Journal*, Am. Water Works Assoc., December, 1927.

† "An Investigation of the Efficiency of the Perforated Pipe Underdrain System in the Washing of Rapid Sand Filters," by Harry N. Jenks, M. Am. Soc. C. E., July, 1920.

flanged access manhole was provided in the common wall between the effluent and pipe galleries at about the mid-point in the length of the wall. At the ends of the galleries next to the Administration Building, sight wells, 4 by 5 ft. in plan, open into the effluent galleries. They permit observation of the filtered water during operation, and provide means of taking out any of the piping in the effluent galleries, in case such a need should arise. The piping in the effluent galleries in conjunction with the strainer system, acts as the collecting system for the filtered water, conveying it from the filters through the controller in the pipe gallery and then back again into the open effluent gallery. The same system of piping in the effluent galleries serves for the distribution of the wash water during the washing process.

The floor of each effluent gallery rests on shale and is designed for a minimum thickness of 10 in. It is reinforced on the bottom with $\frac{5}{8}$-in. square bars, spaced on 6-in. centers and running in both directions. The footings for the columns supporting the effluent gallery roof with its superimposed loads, rest directly on this floor. To transfer the loads to the columns, large beams and girders were provided, those coming under the center line of the filters being 25 in. wide and 52 in. deep. The longitudinal walls of the effluent galleries were designed as slabs tied into the floor and roof. The walls at the extreme east and west ends were similarly designed, advantage being taken of the passive resistance of the earth against the wall. At the Administration Building, due to the lack of passive earth resistance to counteract the load transmitted by the wall to the roof, and to the existence of an expansion joint 73 ft. from the wall, special reinforcement was designed for the roof to transfer the load into the longitudinal walls.

To provide for temperature changes, expansion joints were placed 71 ft. apart. The joints are located immediately below the dividing plane between each group of two filters. The expansion joints extend from north to south across the entire building and from the roof of the effluent galleries down through the foundations. Copper plates were used at all expansion joints to prevent leakage of water. These plates were carried from the bottom of the foundations to the top of the filter tanks.

Pipe Gallery.—The pipe galleries occupy that space in the filter building between the two rows of filter tanks, and extend from the level of the effluent gallery floor to the operating floor. There are two galleries, each having the same length as the effluent galleries. The pipe galleries are 27 ft. wide and approximately 28 ft. high. In these galleries are the settled water conduit and wash-water pipe, with connecting piping for each individual filter, and the filter rate controllers. An 8-in. water-pressure line and a steam-heating line suspended from and approximately 3 ft. below the operating floor, run the entire length of the two pipe galleries and across the Administration Building.

The settled-water conduit is of reinforced concrete, rectangular in section, 7 ft. wide and 8 ft. high. It is supported by concrete columns spaced approximately 9 ft. apart. Expansion joints are placed nearly opposite those in the effluent galleries and 71 ft. apart. All expansion joints are located midway between columns, the conduit accordingly acting as a cantilever. The top of the conduit is at Elevation 237.0, and serves as a walkway along the entire

length of the pipe gallery. This walkway is easily accessible from the floor level in the Administration Building at Elevation 234.0, the top of the transition section of the conduit at the entrance to each gallery serving as a ramp.

The main drain is a rectangular concrete conduit, the top of which is reinforced and serves as a part of the pipe gallery floor. This conduit is designed to have the sides bear directly against shale. It is 5 ft. wide and at the shallowest section, 5 ft. deep. The bottom has a slope of 0.00251 ft. per ft. When washing two filters at the same time, there is a slight upward pressure on a part of the drain roof. Because of this fact the floor drainage from the pipe gallery is not designed to discharge into the main drain, but to empty into a 12-in. cast-iron sub-drain. This sub-drain is laid with open joints, in crushed stone, under the bottom of the main drain, and discharges into it at a point on the slope beyond the filter building, where there is no danger of water backing into the pipe gallery.

The pipe gallery floor is 10 in. thick and rests on the shale. It is of plain concrete, and slopes 3 in. in about 11 ft. from the effluent gallery walls toward the top of the main drain. A vertical drop of 3 in. is made at the junction of the pipe-gallery floor proper with the top of the main drain, thus forming a distinct drainage channel along the longitudinal center of the pipe gallery. This channel discharges into the floor drain sumps, spaced 140 ft. apart. The invert of the drainage channel slopes 3 in. in 70 ft. toward these sumps. These sumps are connected by 6-in. cast-iron pipe to the 12-in. cast-iron sub-drain, previously mentioned.

The wash-water pipe line is a 30-in. cast-iron bell-and-spigot main, located under the settled-water conduit, and supported on concrete beams spanning between the columns supporting this conduit. At the west end of the pipe gallery, the wash-water pipe line is connected to a 24-in. service supply main and this furnishes an additional or emergency source of wash-water supply.

The control of this supply of wash water is by means of an altitude valve. If this valve fails to close when the wash-water tanks are full, the supply is closed by an electrically operated gate-valve between the altitude valve and the wash-water tanks. This emergency valve may be controlled by floats in the wash-water tanks or by a push-button switch. It may also be operated manually by means of a hand wheel. Just outside the building, in a valve vault, is a 24-in. pressure-regulating valve, a 24-in. check valve, and an 8-in. emergency connection to a higher pressure water supply main. The pressure-regulating valve is used to reduce the pressure in the service main to that required in the wash-water system. The 24-in. check valve was required to prevent the water from the 8-in. emergency connection from backing up into the lower pressure 24-in. service main.

The system of piping at each filter consists of a 24-in. influent line controlled by a valve of the same size, a 24-in. effluent line containing a filter rate controller and a 24-in. valve, a 24-in. wash-water line controlled by a valve of the same size, a 30-in. drain line with a 30-in. valve, and an 8-in. filtered-water waste line and valve. All the valves are of the double-disk, parallel-seat type, and all are hydraulically operated except the hand-operated 8-in. filtered-water waste valve.

All valves, 24 in. and larger, where placed horizontally on their sides, are provided with rollers and scrapers. The 30-in. drain valves which have their disks in a horizontal plane, are provided with special heavy bronze mounted slides, operating on bronze tracks built in the body and bonnet of the valve, in order to prevent the wearing of the gate and seat rings. Just before the disks come to a stop, the tracks slope to permit them to drop into a closing position before the wedging mechanism operates.

Special precautions were taken to prevent the scouring of the brass lining by iron rust in the hydraulic valve cylinders. The cast-iron piston, which consists of two rings with a spacer, is provided with cup leathers, and is secured to the stem with bronze lock nuts. The circumference and upper and lower faces of the spacer or piston head, not in contact with the cup leather and followers, are covered with a brass facing, forced into place and securely fastened to the spacer.

The 8-in. pressure line running longitudinally through the pipe galleries, supplies water for operating the hydraulic valves. To avoid shutting down a considerable number of filter units due to a break in this line, it is fed from either end, and valves are placed at the quarter-points. At each end of the line, pressure-regulating valves are provided to reduce the pressure to that required for operating the hydraulic valves. The waste from all the hydraulic valves of each filter is piped to the central gutter of the particular filter to which they belong.

Operating Floor.—The operating floor is at the same elevation as the top of the filter tanks. It is of reinforced concrete, 7 in. thick, with quarry tile surfacing. It is supported on columns resting on the outer walls of the settled-water conduit, and at the filters on ledges constructed as a part of the individual filter tanks. Access from this floor to the top of the settled-water conduit as well as to the floor of the pipe gallery is provided at the ends of the pipe galleries by reinforced concrete stairs.

To provide light and air in the pipe galleries, openings 7 ft. wide by 17 ft. long are located in the operating floor. These openings are centered on the dividing line between adjacent pairs of filter tanks. An ornamental iron rail follows the outline of the openings in the floor and continues, between openings, along the edge of the filter tanks. Easy access from the operating floor to the walks over the filter tanks is provided by means of gates.

Baldwin Reservoir

General Description.—As previously noted, the Baldwin Reservoir was originally planned as a distributing reservoir for the low-service district. The original design was based on a 130 000 000-gal. open reservoir. The excavation was about three-fourths completed when it was decided by the City of Cleveland to construct a filtration plant immediately adjacent to the reservoir, using it as the clear-water basin for this plant. Due to the fact that the excavation was so far advanced, the elevation of the bottom as well as the slope of the floor and the area to be occupied by the reservoir was fixed in a general way.

Baldwin Reservoir, as constructed, is about 1 035 ft. by 550 ft. in plan, and 40 ft. from the floor to the crown of the groined arch roof. The reservoir is divided into two basins of equal size. The maximum depth of water is 36.5 ft. in each basin, and the two basins have a total capacity of 135 000 000 gal. In all there were 1 196 columns, the dimensions of which are shown in Fig. 7.

Conditions Assumed for Design.—Before the actual design of the various parts of the reservoir was begun, a careful study was made of the loads which might be imposed upon them during construction, as well as those which might be expected after the work was completed.

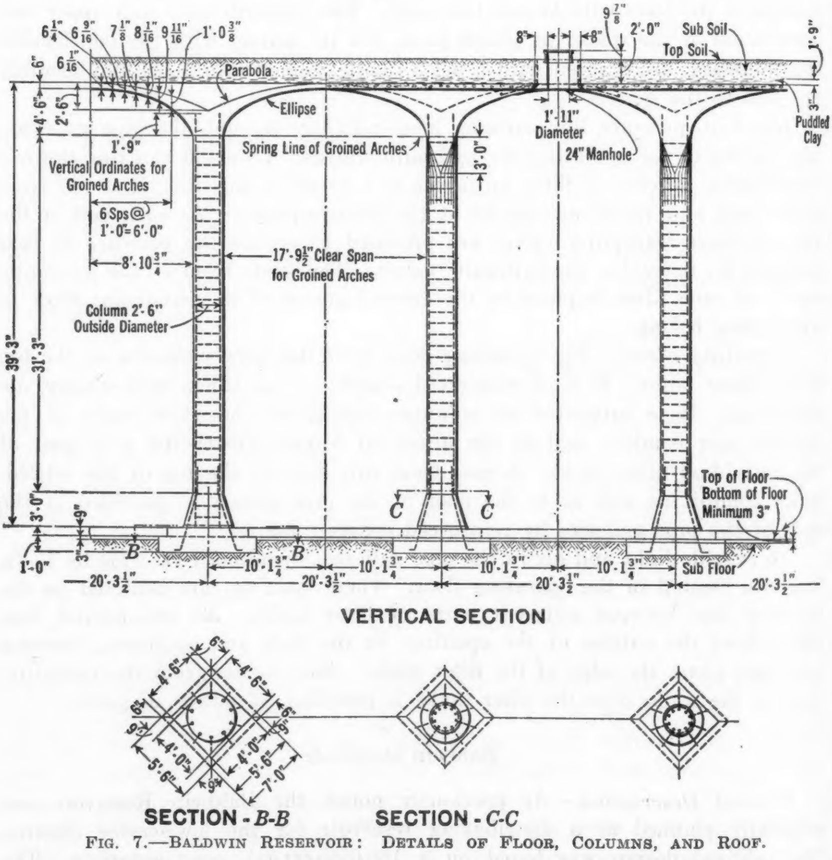

FIG. 7.—BALDWIN RESERVOIR: DETAILS OF FLOOR, COLUMNS, AND ROOF.

The conditions assumed for design before the completion of the reservoir, were:

1.—That the outside walls would be constructed and the back-fill placed against them before the roof arches were built.

2.—That the outside walls would be constructed and the roof arches built before any back-filling was placed against the walls or on the roof.

3.—That the concrete structure would be completed without the back-fill placed against the walls, or on the roof, and that either or both basins would be full of water.

The conditions assumed for design after the completion of the reservoir, were:

4.—That the concrete structure would be completed; that all the back-fill would be placed against the walls and on the roof; that there would be a uniform live load of 100 lb. per sq. ft.; but that there would be no water in either basin.

5.—That the same conditions would exist as are noted for Condition 4, except that either or both basins would be full of water.

In order to provide flexibility in operation and insure circulation of filtered water within the reservoir and also to detect leakage, it was assumed:

6.—That water would be delivered to either or both basins of the reservoir from the City's distribution mains prior to the completion of the Baldwin Filtration Plant.

7.—That water would be delivered to either or both basins of the reservoir from the filtration plant.

8.—That filtered water would be delivered to any or all of the distribution mains leading to the city and to the suction mains leading to the pumping stations from either or both of the basins.

9.—That means would be provided for emptying either basin for the purpose of cleaning or inspecting it and without disturbing the operation of the other basin.

In addition to the special features of design previously mentioned, careful consideration was given to circulation of the filtered water through each basin; to facilities for detecting leakage through the floor and lower sections of the walls; and to ample overflow capacity.

Reservoir Floor.—The floor of the reservoir slopes about 6 in. in 100 ft. from the north and south sides toward the center. The footings of all columns and stiffening walls were made separate from both the sub-floor and the finished floor. The sizes of the column footings were varied to suit the conditions of the shale, the smallest being 5½ ft. square, and the largest, 7 ft. square. In order to prevent the shale from disintegrating, and also to insure a uniform thickness for the finished floor, a sub-floor, having a minimum thickness of 3 in., was placed upon the shale immediately after the excavation was completed. No provision for expansion or contraction was made in the sub-floor. All irregularities in the excavation were filled with concrete.

The finished floor (Fig. 7) is 9 in. thick, and expansion joints are provided on the center lines of columns, namely, 20 ft. 3½ in. apart. The expansion joints consist of strips of prepared asphalt filler, 5 in. wide and ⅝ in. thick, placed at the bottom of the joints, the upper 4 in. being filled with hot asphalt. No steel was used in the finished floor except over the drains.

Columns.—In designing the connection between the tops of the round columns and the bottoms of the groined arches at their springing lines, a departure was made from usual engineering practice, in that the 2 ft. 6-in. round columns were flared out to a 2 ft. 6-in. square, which is the size of the groined arches at their springing lines. The ratio of length of column to

outside diameter is 13.7 and the ratio of length of column to effective diameter is 15.75.

The maximum load on top of each column is about 184 000 lb., and the weight of each column is about 27 000 lb., making a total load of about 211 000 lb. on each column footing. Each column is reinforced with ten ¾-in. square twisted bars, placed 2 in. from the surface of the concrete. These bars are tied together with ⅜-in. round hoops, spaced 1 ft. on centers.

Groined Arch Roof.—Owing to the unusual depth of the reservoir, necessitating very long columns for supporting the roof, considerable throught was given to the most economical type of roof construction. Studies were made of several types of flat-slab design as well as of groined arches of various dimensions. These various designs were carefully compared as to cost, which led to the decision to use a groined arch roof with dimensions as shown in Fig. 7. The average thickness of concrete in the roof above the springing line of the groined arches is $8\frac{5}{16}$ in.

The method of calculating the stresses in the groined arches was based on the hypothesis of least crown thrust, as most generally used in theories of the voussoir arch. The total load on each arch was assumed as acting between vertical planes passed through the groins formed by the intersection of two adjacent arches, these planes intersecting at the center line of columns.

The line of thrust at the crown of the groined arches was assumed as acting through the upper third point. With this condition assumed, the least crown thrust to hold the groined arch in equilibrium under a maximum load of 2 ft. of fill and a live load of 100 lb. per sq. ft., produced a maximum compression of about 225 lb. per sq. in. in the concrete at a section about 6 ft. from the crown. The line of thrust at this point passes through the lower third point. The maximum shear is about 48 lb. per sq. in. under the same condition of loading. The upward pressure of the water did not affect the maximum compression in the arches, as this occurred at a point above the water level; but the maximum shear was slightly reduced.

Another feature in connection with the design of the roof to which special attention is called is the location of the stiffening walls (Fig. 8), which were provided to take the arch thrust in case of failure of a part of the roof. The two basins of the reservoir are each approximately 500 ft. by 500 ft. and the two lines of stiffening walls in each basin divide the roof areas into approximately 250-ft. squares. The location of the stiffening walls does not interfere in any way with the circulation of the water across the basins. Their foundations and the two adjacent column foundations were designed to be poured monolithically, as were also the stiffening walls and the two adjacent columns.

Four ⅝-in. square bars were placed in the crowns of the arches, and sections containing not less than four groined arches were tied together by this steel. Construction joints in the roof were always placed at the crown, midway between columns. In order to prevent surface water from entering the reservoir the construction joints in the roof were covered with burlap and coal-tar pitch.

Barrel Arches.—In designing the barrel arches, the section was made such that the least crown thrust was equal to or less than that of the abutting

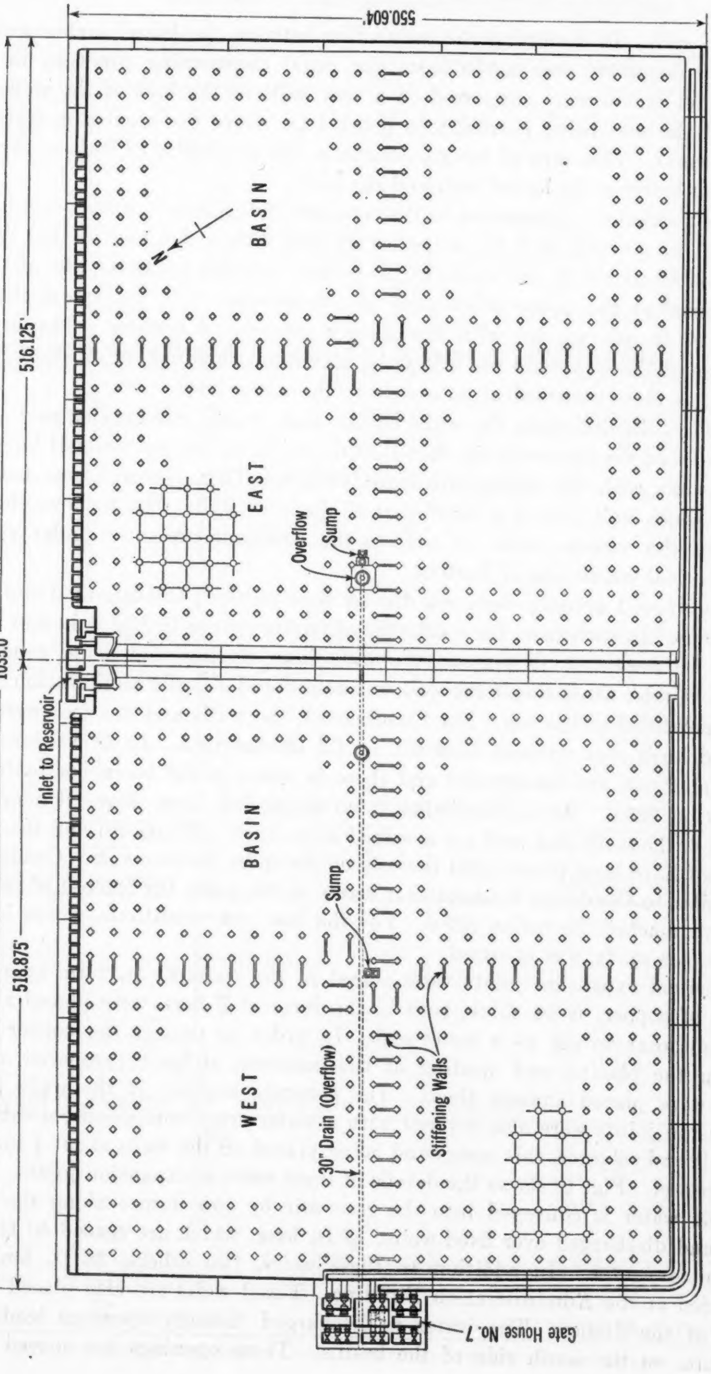

FIG. 8.—PLAN OF FLOOR, BALDWIN RESERVOIR.

groined arch. In designing the connection between the barrel arches and the walls, a departure was made from the usual engineering practice, in that the barrel arches were supported on a seat built in the tops of the walls, the back of the seat being carried to a point 3 in. above the maximum elevation of the water. This type of design eliminates the possibility of leakage through the joint between the barrel arch and the wall.

The maximum compression in the concrete of the barrel arches when loaded with fill to a depth of 2 ft. at the crown and with a live load of 100 lb. per sq. ft., is about 130 lb. per sq. in. at the crown, at which point the line of thrust is assumed at the upper third point of the section. The maximum shear is about 17 lb. per sq. in. with the same condition of loading. The upward pressure of the water did not affect the maximum compression or shear in the arches, as these occurred at points above the water level.

Walls.—In designing the walls on the east, north, and greater part of the west sides of the reservoir, the fact that the backs of the walls would be placed against the rock was taken into consideration. This, however, was not true of the south wall, nor of a small part of the west wall. Fig. 9 shows the outlines of the various walls, as well as the resultant pressures under various hypothetical conditions of loading.

Condition 1 actually occurred during construction; Conditions 2 and 3 did not occur. In each case, the coefficient of friction given in Fig. 9 is that which would be necessary to prevent the wall from sliding under the conditions noted. Under Condition 1 the wall was considered as built to Elevation 229.25 and back-filled to the top. For Condition 2, the walls and the roof were considered built, but without back-fill, or fill on the roof. In Condition 3 the walls and roof are constructed and there is water in the basin (or basins) to Elevation 225.0. As in Condition 2, no filling has been done. Under Condition 4, the walls and roof are assumed to be built, all back-fill and the fill on the roof have been placed, and there is no water in the reservoir. Condition 5 is similar to Condition 4 except that water in the basin (or basins) is assumed to have reached Elevation 229.0. For the last two conditions, a live load of 100 lb. per sq. ft. was assumed.

Vertical expansion joints were placed in the walls 81 ft. 2 in. apart, and strips of copper, $\frac{1}{16}$ in. thick, bent in the form of Z-bars, were placed at each vertical joint to act as a water-seal. In order to protect the copper strips during the placing and spading of the concrete, strips of prepared asphalt filler were placed against them. The concrete surface of the walls at the expansion joints were also covered with a water-proof compound consisting of asphalt and asbestos, this compound being placed on the walls about $\frac{1}{2}$ in. thick by a trowel. Fig. 10 shows the details of these vertical expansion joints.

The water is conveyed into the reservoir by two flumes along the north wall and discharged over fixed weirs, 13 ft. long, which are spaced 40 ft. 7 in. center to center. In addition to these weirs, two others, 15 ft. long, are provided at the Administration Building. Fixed weirs are also placed at the ends of the flumes. The water is discharged through openings leading to conduits on the south side of the basins. These openings are spaced 10 ft.

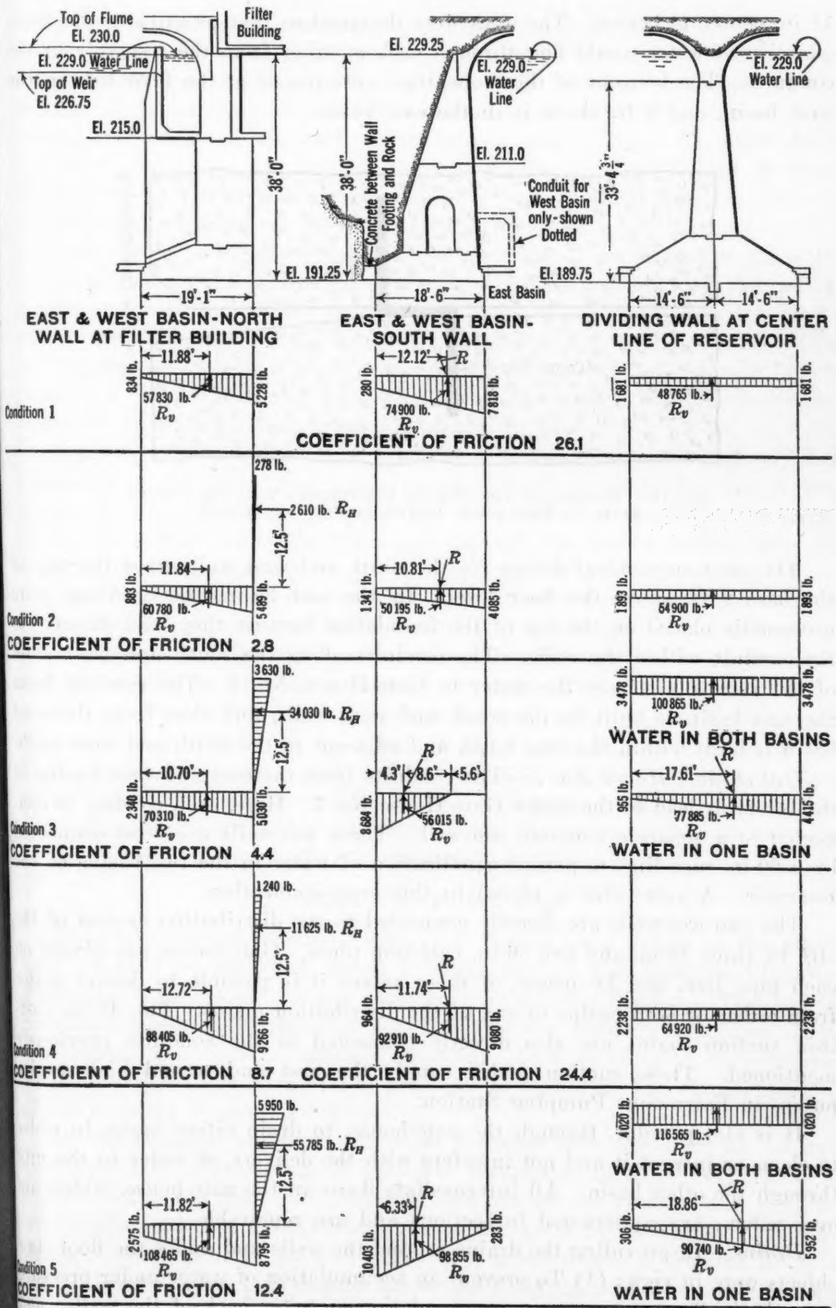

FIG. 9.—OUTLINE OF WALLS AND DISTRIBUTION OF FOUNDATION LOADS.

1¾ in. center to center. The sizes were designed so that practically the same quantity of water would flow through each opening from the reservoir to the conduits. The bottoms of these openings were placed at the floor line in the west basin, and 2 ft. above it in the east basin.

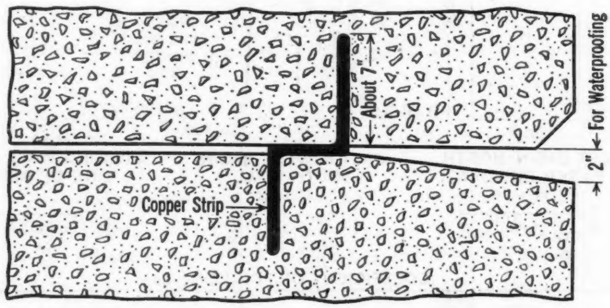

FIG. 10.—SUGGESTION FOR PLACING CONTINUOUS VERTICAL COPPER STRIP AT EXPANSION JOINTS ON FACE OF WALLS.

The most economical design for the south and west walls fixed the top of the base 2 ft. above the floor level. In the east basin, the openings were necessarily placed on the top of the foundation because they lead directly to the conduit within the walls. The conduits along the south and west sides of the reservoir convey the water to Gate-House No. 7. The conduit from the east basin is built in the south and west walls, and that from the west basin is built within the west basin and adjacent to the south and west walls.

Outlet Gate-House No. 7.—The conduits from the east and west basins of the reservoir lead to the outlet Gate-House No. 7. Here, each conduit is connected to a separate concrete wet-well. These wet-wells are cross-connected by a 60-in. pipe line, to permit equalization of water in the two basins of the reservoir. A gate-valve is placed in this cross-connection.

The two wet-wells are directly connected to the distribution system of the city by three 48-in. and two 36-in. cast-iron pipes. Gate-valves are placed on each pipe line, and by means of these valves it is possible to deliver water from either or both basins to any of the distribution mains. Two 48-in. cast-iron suction mains are also directly connected to the conduits previously mentioned. These suction mains supply the first and second high-service pumps in Fairmount Pumping Station.

It is also possible, through the gate-house, to drain either basin, in order to clean or inspect it and not interfere with the delivery of water to the city through the other basin. All intermediate floors in the gate-house, which are over valves, are constructed in sections and are removable.

Drains.—In providing the drains around the walls and under the floor, two objects were in view: (1) To prevent an accumulation of water under pressure beneath the floor and excessive accumulation of water back of the walls; and (2) to detect and locate leakage easily.

It is obvious that an accumulation of water back of the walls either from leakage or ground-water might produce an upward thrust on the floor when the water in the reservoir is lowered below the level of the water on the outside of the walls. Drains were provided beneath the reservoir floor, around the outside of the walls at the bottom, and at about Elevation 211.0. All these drains, except parts of the one at Elevation 211.0 lead to Gate-House No. 7, where they discharge into a sump. Each drain is properly marked in the gate-house, so that it would be possible to determine the approximate location of any leakage or accumulation of water behind the walls.

Particular attention is called to the detail in connection with the outside drains at the bottom of the east and north walls. These drains are placed within the walls, and are all cast-iron bell-and-spigot pipe with leaded joints. At varying intervals (not more than about 80 ft.) pockets, filled with broken stone, are provided in the rock backing of the walls, and the drains are connected to them by short pieces of cast-iron pipe.

Overflow.—Each basin is provided with an overflow and a sump, each connected to the 30-in. cast-iron pipe drain which leads to Fairmount Reservoir. The sump connections are for the purpose of emptying the basins of all water that cannot be taken out through the drains connected to the conduits at the gate-house.

Ventilation.—In order to provide ventilation for the reservoir, fifty-six 24-in., four 36-in., and two 48-in., ventilating manholes were placed in the roof. Those along the north side were placed over the weirs of the flume through which the water flows from the filtration plant.

Fill on Roof.—The concrete roof is covered with a layer of thoroughly compacted clay puddle, 6 in. thick at the crowns of the groined arches. This, in turn, is covered with 1 ft. of sub-soil and 6 in. of top-soil. The entire top of the reservoir will be seeded with grass. Because the reservoir slopes from the north and south sides toward the center, it was necessary to provide a drain to carry off any accumulation of surface water.

Alum Storage Building

The alum storage building (Fig. 1) is located on a switch of the Cleveland Short Line Railroad, about ½ mile west of the Baldwin Filtration Plant and adjacent to the new Fairmount Pumping Station. The lack of railroad connection to the chemical house of the Baldwin Filtration Plant necessitated this separate storage building. This building is of reinforced concrete and is 51 ft. by 87 ft. in plan and 43 ft. high. The storage capacity of the building is about 1 000 tons.

In the building are four reinforced concrete bins supported on columns (see Fig. 11). The bottom of each bin terminates in three hoppers, 16 ft. square at the top with 15-in. square openings in the bottom, and equipped with single undercut gates through which the alum is discharged. The total depth of the bins is 17 ft. 6 in. The hopper sides have a slope of 50° with the horizontal. The bins extend across the building, and a common dividing wall separates one bin from another. The entire area under the bins is used for truck runways.

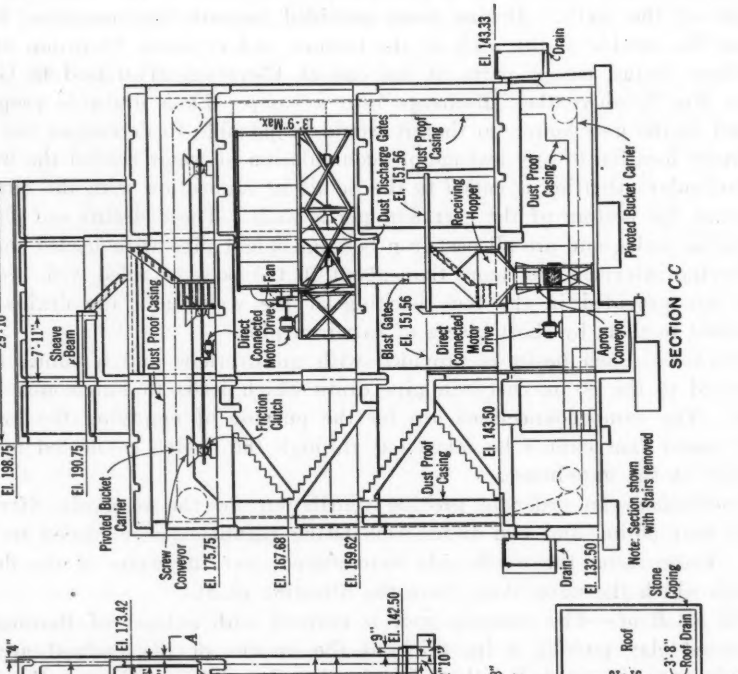

FIG. 11.—GENERAL SECTIONS, ALUM STORAGE BUILDING.

Construction

The construction of Baldwin Filtration Plant was carried on under general contracts for the different parts of the work. The construction operations will be described in the order in which they were done.

Baldwin Reservoir

In the winter of 1914-15, the City, by direct labor, began preliminary work by stripping the top-soil from the site of a proposed 130 000 000-gal. open reservoir. On April 9, 1915, a contract was awarded for the excavation of this reservoir. The contractor had completed about three-fourths of the work when he encountered financial difficulties and, as a result, suspended operations in July, 1918. All the excavation remaining to be done was in solid rock and, in consequence, the City entered into a "cost-plus" agreement with a quarrying company to complete the work. Excavation was resumed in the winter of 1918-19, and completed in 1920, except for final trimming.

During the period covered by the excavation work, the location of the proposed second filtration plant, as explained in the Historical Section, had been changed to the Baldwin site. Accordingly, additional land was purchased on either side of the reservoir, and this extended the area of city-owned land to the streets surrounding the site.

Bids were received and a contract for the construction of the reservoir was entered into on May 27, 1921; but objection was taken to this award and it was annulled by the Courts. The work was re-advertised, bids were received again, and the contract was awarded to the low bidder on November 18, 1921.

While the contractor was assembling his construction plant, he began the final trimming within the reservoir, and the final excavation for the wall footings. Due to the disintegration of the shale on exposure to air, it was necessary to leave the final trimming of all excavation until just prior to placing the concrete.

Pending the erection and completion of the central concrete mixing plant and the Lidgerwood 15-ton cableway, the contractor erected a temporary mixing plant within the reservoir excavation, near the south wall. Concrete mixed at this plant was hoisted to the top of a steel tower and distributed by chutes. By this means concreting of the south wall was commenced in the spring of 1922.

After the completion of the central mixing plant and the cableway, the temporary mixing plant was dismantled. After this, all concrete was mixed at the central mixing plant, and distributed by the movable cableway. The cable spanned the work in a north and south direction and was capable of movement in an east and west direction over the entire length of the reservoir. Work on the floor, column foundations, and columns, and on the groined arch roof, was started along the east wall and progressed toward the west.

Work was continued night and day during 1922, until the winter weather stopped operations in December. At this time the entire roof of the east

basin had been completed and a good start had been made on that of the west basin. Concreting was resumed in the following April, and the entire work, except for the completion of the rough grading, was completed in the late summer of 1923. Fig. 12 is a view of the construction work and Fig. 13 shows an interior view of the completed structure.

Construction Plant

Cableway.—The layout of the construction plant is shown on Fig. 14. The 15-ton cableway was electrically driven. All operations of carriage, lifting hook, and both towers were controlled by one operator on the head tower. The main cableway hoist motor was an induction motor (440-volt, 300-h.p., at 950 rev. per min.). The motors for moving the head and tail-towers longitudinally along the tracks were considerably smaller (440-volt, 75-h.p., at 1 150 rev. per min.). The main cable was 2½ in. in diameter. The head and tail-towers were of wood construction, 85 ft. high, and all timbers were of full length without splices. The cableway towers traveled on standard freight-car wheels and axles. The track was composed of five 100-lb. rails and with a distance between the first and fifth rails of 40 ft. 2 in. The cableway was capable of handling a 7-cu. yd., bottom-dump, concrete bucket.

Central Concrete Mixing Plant.—The concrete mixing plant (see Fig. 14) was 30 ft. by 63 ft. in plan and 55 ft. high. It was of wood construction, except that the beams supporting the overhead material bins were of steel. The coarse and fine aggregate was delivered to the mixing plant in hopper-bottom cars of standard gauge, which were dumped from a material trestle, 250 ft. long and 22 ft. high. The material was lifted from the receiving hoppers beneath the trestle to the bins above the batch-measuring hoppers by three bucket elevators, 90 ft. long. The buckets were mounted on a 15-in. belt. Each elevator had a capacity of 75 tons per hour. Two of the bucket elevators were used for conveying stone and one for sand. Each elevator, together with its feeder conveyor, was driven by a 20-h.p. motor.

Cement was delivered in sacks in cars on a track ending at the east side of the mixer building. This track was located at such an elevation that the floor of the cement cars was on a level with the operating and cement storage floor of the mixer building. Platforms were provided along both sides of the cement tracks so that the cement could be moved by warehouse trucks directly from the car to the mixers or to storage.

The two hopper-bottomed overhead stone bins had a capacity of about four cars each, and the one similar bin for sand had a capacity of about six cars. The sand bin was located between the two stone bins. Concrete was mixed in two 1-yd. mixers, driven by 30-h.p. motors. Material was drawn by gravity from the bins directly into bottom-dumping steel measuring boxes. Water was supplied from measuring barrels, equipped with gauge glasses. Each set of measuring boxes discharged into a steel receiving hopper, which, in turn, emptied into the body of the mixing drum. Cement was dumped from bags through a grating in the floor of the storage room directly into this receiving hopper. As a result of this arrangement, the sand, stone, cement, and a measured quantity of mixing water were supplied to the

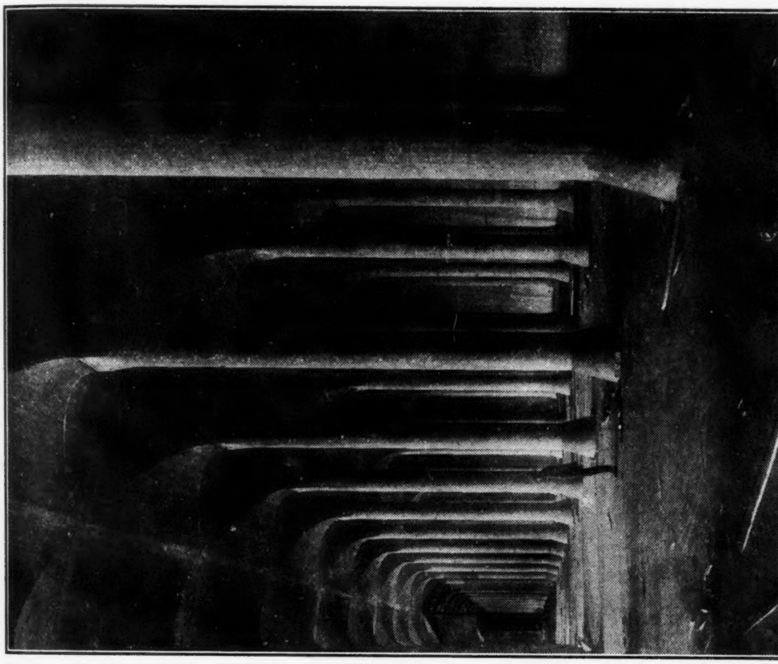

FIG. 13.—INTERIOR VIEW OF BALDWIN RESERVOIR.

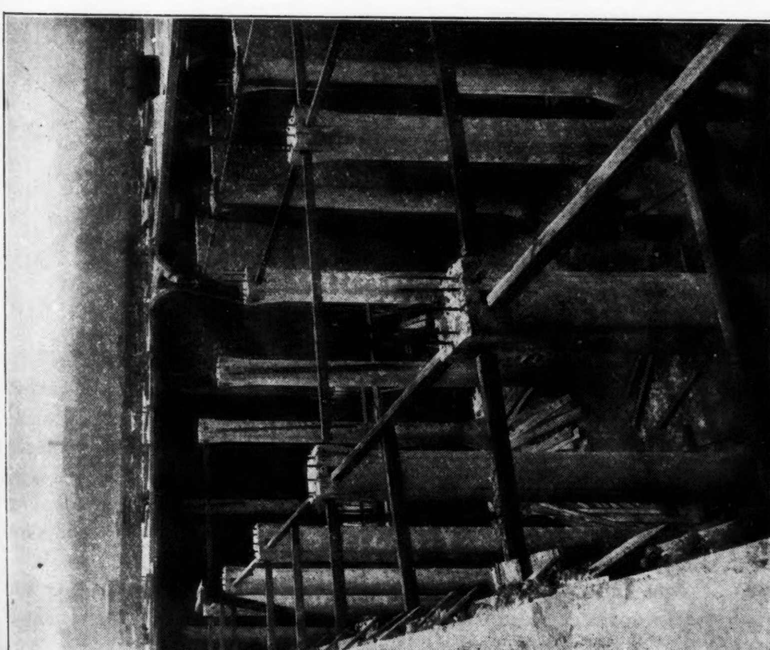

FIG. 12.—VIEW SHOWING BRACING AT TOPS OF CONCRETE COLUMNS, PREVIOUS TO PLACING FORMS FOR ARCHES, BALDWIN RESERVOIR.

mixer simultaneously. The mixers were discharged into a common conical steel hopper with a capacity of about 9 cu. yd. A slide-gate at the bottom of this hopper permitted the concrete to be discharged into the 7-yd. concrete bucket as needed.

Placing Concrete.—The concrete was poured from 7-yd. buckets. After the buckets were filled they were transported on flat cars running on narrow-gauge (36-in.) tracks. Locomotives hauled the loaded cars to a position under the cableway, which had been previously moved so as to span over the form to be filled with concrete. An emptied bucket, hanging on the cableway hook, was placed on an empty flat car, and the full bucket of concrete was then picked up by the hook and moved to the proper position directly over the form and lowered to position for pouring. During the time that the bucket was being moved along the cable, the empty bucket was on its way to the mixer for refilling.

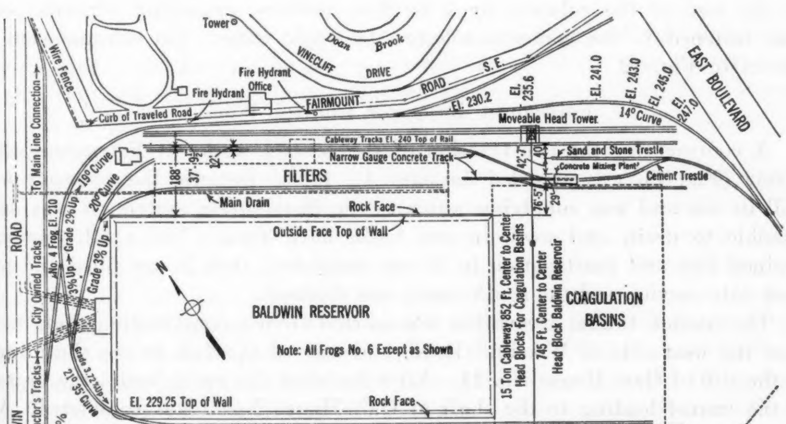

FIG. 14.—COAGULATION BASINS AND FILTERS, CONSTRUCTION PLANT LAYOUT.

Forms for Concrete.—Considerable study was given to the design of forms, in order to keep costs as low as possible. Practically all forms were made in separate panels so as to permit their use many times. Wall-form panels were made of various sizes, depending on their intended use. The largest single panels were about 20 ft. high and 45 ft. long. Forms for the groined arch roof were made in four sections for the part supported by one column.

Except for columns, all forms were made of ⅞-in. center-matched lumber, nailed to 2 by 6-in. studs which were spaced on 16-in. centers. Vertical forms were braced by 4 by 6-in timbers running horizontally outside the

studding and held from spreading under the force of the horizontal thrust of the wet concrete, by $\frac{5}{8}$-in. bolts running through the section to be concreted. These bolts were made up with pump-rod couplings, spaced 4 in. inside of each form face. These horizontal timbers were spaced a variable distance apart, depending on their elevation above the bottom of the form. Column forms were made of steel, each complete form consisting of two halves. All forms were handled by the cableway.

Due to the great depth of the reservoir, unusual methods were used in supporting the forms for the groined arch roof. The entire load of the roof forms as well as of the concrete, was carried by wood and steel collars fastened around the tops of the previously poured columns, and by eight wooden posts, each 8 in. square, extending from floor to roof, and located around the perimeter of the groined arch unit supported by one column. One post was located at each of the four corners, and additional posts midway between the corner posts. These posts were braced in a horizontal plane at the tops of the columns by 4 by 6-in. timbers, extending between posts and fastened to the concrete columns by angle clips. The arrangement is shown in Fig. 12.

Fairmount Reservoir Control Works

A contract for the construction of the control works at Fairmount Reservoir (Fig. 1), was awarded January 15, 1923. Because the reservoir was still in use and was supplying water to the distribution system, it was only possible to drain and work in one basin at a time. The north one was drained first and construction in it was completed; then it was filled and put back into service and the south basin was drained.

The suction tunnel excavation was started from a construction shaft sunk near the west side of Woodhill Road, and carried through to the shaft sunk at the site of Gate-House No. 11. After draining the south basin, the branch of the tunnel leading to the shaft at Gate-House No. 9 was completed. All the tunnel excavation was in shale, although there was some variation in its hardness. A short section of the main tunnel was in material so soft as to require timbering, whereas the greater part of each of the branch tunnels was in very hard shale.

Removable steel-plate forms were used in the tunnel except at transition sections, where wood forms were employed. Concrete for the tunnel lining was mixed at the top of the construction shaft in a drum mixer and was transported to the point of placing by a pneumatic concrete blower, which forced the mixed concrete through a 4-in. steel pipe line.

Coagulation Basins

Excavation.—The excavation for the coagulation basins was done under a contract, bids for which were received April 29, 1920. The contractor began work June 25 of the same year and continued working until March 8, 1921. The area covered by the basins was excavated to an elevation 1 ft. above the base of the proposed wall foundations. The material encountered was clay and soft shale. Steam shovels loaded the excavated material into

dump trucks and dump wagons, which removed it to storage and disposal piles.

Three steam shovels were used regularly on the work. The maximum number of dump trucks used in any one day was twenty-three and the maximum number of teams was thirty. Progress of the work was greatly delayed in wet weather, due to the inability of the trucks and teams to maneuver in the wet clay.

The total material excavated was approximately 117 000 cu. yd. About 20 000 cu. yd. suitable for back-filling were hauled to a storage pile across Doan Brook and about 1¼ miles east of the site of the work. An additional 10 000 cu. yd. were used in completing a fill along the south side of Doan Brook, and in the landscaping plans in the development of the valley of this brook. An additional 30 000 cu. yd., suitable for back-filling, as well as about 55 000 cu. yd. intended for use as clay puddle on the roof of the filtered-water reservoir, were stored between the site of the basins and the boulevard to the east.

Due to delay on the part of the City in acquiring some of the land needed for the site of the filters—the general excavation of which was also included in this contract—it was necessary to postpone the excavation of the depressed portions of the four coagulation basins, as well as the excavation for the filters.

Construction.—A contract for the construction of the coagulation basins was awarded June 1, 1923. The contractor who was just completing the Baldwin Reservoir, was the low bidder on the contract for this work. This made possible, by some re-arrangement, the use of much of the old construction plant on the new contract. (See Fig. 14.)

Because the coagulation basins were located along the east side of the reservoir and extended some distance beyond its north and south limits, a direct extension eastward of the existing cableway tracks would bring them inside the north and south ends of the basins. The contractor accordingly moved the head-tower tracks to a new position, parallel to the old, and north of the existing concrete mixing plant, and extended the track to the easterly limit of the coagulation basins. The mixing plant had been purposely located just outside the north limit of excavation. Insufficient room for the tail-tower tracks, between the south wall of the basins and Woodstock Avenue, prevented the location of these tracks so that the cableway carriage could span the full length of the basins. However, the existing tail-tower tracks were extended east over the south end of the basins for their full width. This arrangement placed the entire coagulation basin area, except the part at the southerly end, within working position of the hoisting hook on the cableway carriage. The increased span necessitated the substitution of a 5-cu. yd. bucket in the place of the 7-cu. yd. bucket used on the reservoir, thus reducing the capacity of the cableway to about 12 tons.

After completing the re-arrangement of the construction plant, the contractor excavated for and placed the concrete foundation for a considerable length of the by-pass conduit. At the same time a ¾-yd. caterpillar steam shovel was used in excavating the depressed portion of Basin No. 1. When

considerable of this basin had been excavated to within a few inches of the final grade, a steam channeling machine was set up over the location of the longitudinal drain, and channel cuts were made along each side. After excavating, this method left the walls of the drain in good condition to receive the concrete lining, using only interior forms and depositing the concrete directly against the shale through which the channel cuts had been made. This practice of making cuts through the shale and rock and depositing concrete directly against the vertical surface was typical for similar conditions throughout the work. It was of particular value in situations like the one before mentioned, where no back-filling could be permitted behind the drain walls on account of possible settlement and resultant cracking of the basin floor.

At the same time that the longitudinal drain was being built, foundations for the dividing wall between Basins Nos. 1 and 2 and the column foundations for Basin No. 1 were constructed. After concreting the drain walls, dividing wall, and column footings for a sufficient length of basin, the last few inches of shale were removed from the general floor area and 4 in. of sub-floor concrete were placed. This concrete formed a working base on which the steel for the finished floor could be placed, and insured that the full depth of concrete for this floor would be free from disintegrated shale. If the sub-floor had been omitted, it would have been possible for serious disintegration of the shale to have occurred during the placing of the floor steel. This shale might have worked up into the floor concrete while it was being placed, thus forming weak spots.

At all construction joints in the floor-slab, as well as where the floor concrete joined the column and wall foundations, particular care was taken to insure water-tight joints. Before pouring the floor, strips of prepared asphalt filler $\frac{5}{8}$ in. thick were placed at the bottom of the joints. These extended to within 4 in. of the floor surface. In the upper part of the joint, flat steel plates were placed. They were withdrawn after the concrete had set, and the space was filled with melted asphalt. This type of joint was used throughout the work in locations requiring a water-tight floor joint. In addition, all vertical joints in the floor and sub-floor were staggered at least 12 in.

At the transverse joints in the wall and conduit foundations, copper strips provided adequate water-stops, and the poured asphalt joints were not required. In order to provide room for expansion at these joints, the existing concrete surface was coated with about $\frac{1}{2}$-in. layer of an asphaltic fiber cement. This provided a cushion, which could be compressed by the expanding concrete and would prevent spalling of the concrete edges or displacing of the sections due to temperature changes. At vertical construction joints in the walls subjected to water pressure, a copper strip—attached at its lower end to the horizontal copper strip in the foundation—served as a water-stop, and provisions, the same as previously outlined, were made to take care of temperature changes.

Only a few sections of the finished floor in Basin No. 1 and even less in Basin No. 2 had been completed, when concreting was stopped for the winter

of 1923-24. The remainder of the year 1923, as well as the first few months of 1924, were used in completing the excavation of the depressed part of Basin No. 1, as well as the depressed parts of the other three basins. This excavated material was loaded on standard gauge dump cars and placed in a railroad fill some distance from the site of the work.

Early in the spring of 1924, the contractor started making forms for the columns and for the groined arch roof of the basins. (See Fig. 15.) For the sections for the arched roof 596 forms were made, or enough to permit pouring the arches over 149 columns at one time. This required the use of each form on an average of seven times during the course of the work.

Concreting in the groined arch roof was commenced in Basin No. 1 on April 26, 1924. The columns were poured for the full height in one operation, and then, before commencing on the arches, a sufficient time was allowed to elapse to permit the concrete in the column forms to shrink. The arches were poured in groups of four, and usually three of these groups were completed in one pouring, making a double row of arches from one barrel arch to the next. Column and arch forms were left in place 14 days in the spring and 10 days in the hot summer weather. The arches were partly loaded after 21 days.

On November 1, 1924, the work in Basin No. 4 had been completed as far south as the running track in front of the tail-tower. The tail-tower tracks were then removed from the easterly half of this basin and construction work in this area carried on by means of a 35-ton locomotive crane, operating from the running track in front of the tail-tower tracks, and also from the track just outside the south wall. As soon as the eastern half of this end of the basin had been completed, the remaining tail-tower tracks in this basin were removed and operations carried on by a locomotive crane, the same as for the eastern half of the basin. In this same manner the other basins were completed until the eastward travel of the cableway was stopped at the westerly side of Gate-House No. 1.

Filters and Effluent Galleries

Excavation.—The general excavation of the Administration and Filter Building site was included with the excavation for the coagulation basins, for which bids were received April 29, 1920. Delay by the City in acquiring the land on which these structures were to be located prevented the immediate excavation, and the work was assigned to the contractor having the general construction of Baldwin Reservoir. Excavation of this site was begun in the late summer of 1923.

Construction.—On October 24, 1923, bids were received for the construction of the Administration and Filter Building, and the contract was awarded to the low bidder, who was already engaged in constructing the coagulation basins.

By extending the cableway tracks west (Fig. 14) for the full length of the filter building, the same plant could be used as for the coagulation basins. The contractor immediately began excavating the 12-in. cover left on the foundation bed and prosecuted the work so vigorously, that when con-

crete operations were suspended near the end of December, 1923, the concrete floor-slab for the southeast effluent gallery, as well as two-thirds of the walls around this same section, were completed.

During the remainder of the winter, the excavation of the main drain was completed. The material encountered was composed of layers of a hard, fine-grained sandstone, between which were thin layers of hard shale. The sides of the main drain were cut with a channeling machine, and the lateral connections, located at every filter, were cut by drilling closely spaced holes, with jack-hammers.

Concreting was resumed early in the spring of 1924. The roof-slab over each effluent gallery was poured in five sections. The construction joints were placed at the dividing plane between the double beams located at the end of every second filter. Due to the length of time required in pouring each roof section of the effluent gallery and the great amount of beam steel located just above the columns, it was found best to pour all the columns in any one section as soon as the column and beam forms were completed, but before the beam and slab steel had been placed. The columns were poured up to the under side of the deepest beam supported by that column. Before placing the forms and concreting the last roof section in each gallery, all the pipe and fittings for the collector system were lowered into the gallery.

Each of the concrete filter tanks (Fig. 16), including the floor, walls, and top walks, was constructed in one operation. Seven sets of forms, and four additional sets, complete except for the outside forms for the two side walls, were prepared. This permitted work on eleven filter tanks at one time, as each of the four sets of forms lacking the outside wall forms, could be used between two filter tanks already poured and stripped.

Each filter tank was poured as a monolith, and the first step was to pour the floor. This served to hold the mortar in the concrete in the side walls and prevented its escape under the lower edge of the inner supported wall form.

The quantity of concrete in one filter tank was 156 cu. yd. The time required to concrete one filter ranged from 3¼ to 5 hours, the general average being a little more than 4 hours. The first filter was concreted on May 12, and the last one August 29, 1924.

Chemical House and Mixing Flumes

Bids for constructing the chemical house and mixing flumes were received on July 23, 1924. These were rejected, the work was re-advertised, and new bids were received on August 20. The contract was awarded to the contractor already engaged in building the remainder of the plant.

Because the site of the work was partly covered by the existing cableway tail-tower tracks, it was necessary to restrict the westerly movement of the cableway to a point about midway of the work under this contract. The cableway tracks were then removed west of this point and excavation for the chemical house was begun.

Standard-gauge railroad tracks were laid along either side of the building site and locomotive cranes operating on these tracks were used to hoist

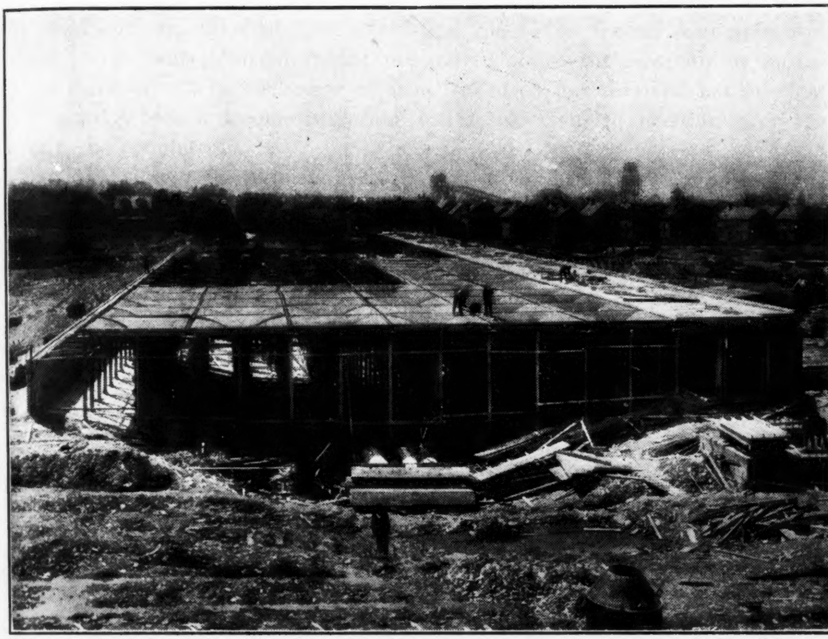

FIG. 15.—VIEW SHOWING FORMS FOR GROINED ARCH ROOF OF COAGULATION BASIN, BALDWIN FILTRATION PLANT.

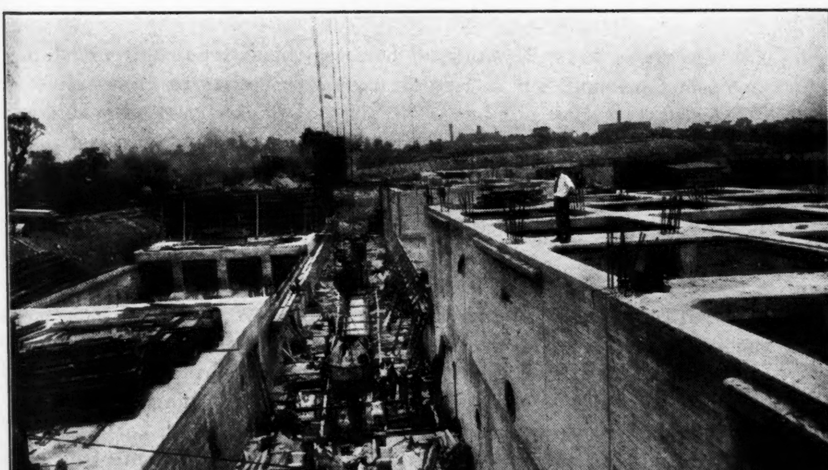

FIG. 16.—PIPE GALLERY AND FILTER TANKS DURING CONSTRUCTION, BALDWIN FILTRATION PLANT.

the excavated material and to move and place the forms. Concrete was mixed in the central mixing plant and moved across the reservoir by means of the cableway. The buckets were then loaded on flat cars and hauled close to a point where a locomotive crane could lift them to position over the form to be concreted.

By the time the westerly half of this work had been completed, the cableway was dismantled and a track was laid from the mixing plant around the east and south sides of the coagulation basins to connect with the working track on the south side of the chemical house. Concrete for the remainder of this work was brought from the central mixing plant by means of this track. The dissolving and solution tanks, as well as the small orifice boxes were each poured in one operation up to a construction joint above the level to which they would be filled. This eliminated the possibility of leakage, which might have occurred, had there been a construction joint where the wall and floor joined.

To guard against the disintegration of concrete in the tanks to be used for alumn solution, a richer mixture was used in order to obtain as dense a concrete as possible. For the dissolving and solution tanks, with 12-in. walls, concrete proportions of approximately 1:1.5:3 were used, and for the orifice tanks, with walls 6 in. thick, a mixture of approximately 1:1.25:2.5 was used. In addition, the interior surfaces of all these tanks were coated with a special acid-resisting paint.

Filter Equipment

Bids for filter equipment, received on June 12, 1924, were rejected, and the work was re-advertised as a separate part of the chemical house contract. New bids were received on July 23, 1924, and the work was awarded to the low bidder, who immediately started manufacturing the equipment.

The cast-iron manifolds and laterals, as well as the effluent piping and rate controllers, were delivered to the site of the work in railroad cars. Delivery of the rate controllers, as well as of the filter manifolds, was started on October 8, 1924.

By January 30, 1925, the superstructure over the filters and operating gallery had so nearly reached completion that the contractor for the filter equipment was permitted to start placing the filter rate controllers. On February 10, the first manifolds were placed in position in the filters.

The large number of filters and the repetition of the same steps at each filter permitted doing the work in a very rapid and efficient manner. It was subdivided into different operations, each of which was performed by a separate crew. As an instance of this subdivision, the work inside the filters may be cited. One crew confined its efforts to placing in position and bolting down the manifolds, and then moving on to the next filter as soon as this operation was finished. Another crew followed and drilled the holes in the floor by means of which the lateral chairs were anchored in position. The drillers were followed by a crew which placed the laterals and chairs in position, and this work was followed by the various operations, such as pour-

ing the lead joints, caulking joints, placing brass strainers on the manifolds, placing the various layers of gravel, and, finally, placing the filter sand.

The filter gravel and sand were purchased by contract. The first bids were rejected and new bids were received November 6, 1924. The coarse filter gravel, ranging from $1\frac{1}{2}$ in. to $\frac{3}{4}$ in. in size, was obtained from the Ohio River, near East Liverpool. The gravel ranging from $\frac{3}{4}$ to $\frac{1}{4}$ in. in size, a part of the gravel ranging from $\frac{1}{4}$ to $\frac{1}{8}$ in. in size, as well as part of the gravel ranging from $\frac{1}{8}$ to $\frac{1}{16}$ in. in size, was obtained from Copley, Ohio. The remainder, as well as the filter sand, came from Phalanx, Ohio.

All gravel and sand was received in railroad cars on tracks laid close to the filter building. The three larger sizes of gravel were unloaded by shoveling on to a belt conveyor, which carried it inside the building and dumped it into a hopper from which it could be drawn into wheel-barrows. The smallest size of gravel, that from $\frac{1}{8}$ to $\frac{1}{16}$ in., and all the sand were unloaded directly from box cars into the filters by means of ejectors placed in the cars.

The following approximate quantities will give some idea of the amount of work involved in constructing the Baldwin Filtration Plant:

Excavation, in cubic yards	1 100 000
Fill, in cubic yards	290 000
Concrete, in cubic yards	185 000
Barrels of cement used	290 000
Brick	2 300 000
Cut stone, in cubic feet	102 000
Reinforcing steel, in pounds	10 900 000
Structural and miscellaneous steel, in pounds	3 000 000
Cast-iron pipe and fittings, in pounds	8 000 000
Filter sand, in cubic yards	5 400
Filter gravel, in cubic yards	3 500

These quantities do not include work on the alum storage building, the Fairmount Control Works, the pipe lines leading to the filter plant, nor the pipe lines leading from Gate-House No. 7, that connect into the distribution system. Fig. 17 is a panoramic view of the finished plants.

General Equipment

The equipment of the Baldwin Filtration Plant is best described under six main divisions: The alum storage plant, chemical house, administration and filter building, and lighting, heating, and telephone system.

Alum Storage Plant.—To facilitate the unloading of alum from the box cars in which it is received, a power shovel or car plow, operated by a 5-h.p. motor at 1 200 rev. per min., is provided. The alum received from the car passes through a chute to an apron feeder conveyor driven by a 2-h.p. motor at 1 200 rev. per min. This conveyor can deliver the alum either to a 24 by 20-in. rigid hammer crusher, operated by a 20-h.p. motor at 900 rev. per min., or directly to a pivoted bucket carrier. The crusher has a capacity of 20 tons per hour, when crushing 5-in. lump alum to approximately $\frac{1}{2}$-in. pieces.

The alum is lifted to the top floor of the building by a pivoted bucket carrier traveling at the rate of 40 ft. per min., and driven by a 5-h.p. motor at 1 200 rev. per min. The buckets are 18 by 15 in. and, after moving

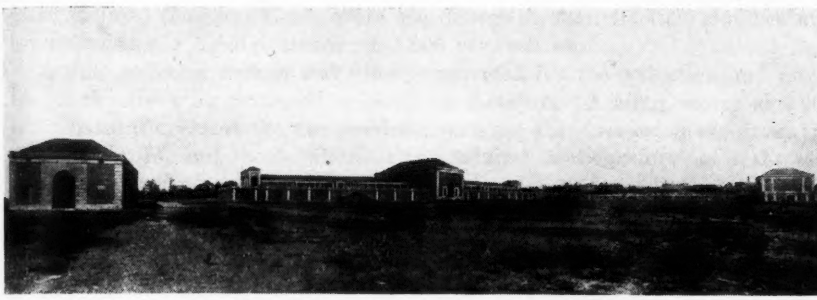

FIG. 17.—VIEW OF BALDWIN FILTRATION PLANT.

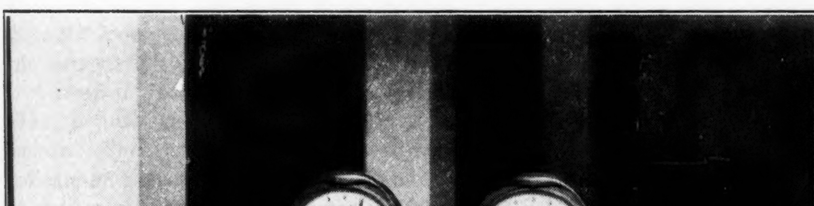

FIG. 18.—ONE OF THE BRONZE AND MARBLE FILTER OPERATING TABLES, BALDWIN FILTRATION PLANT.

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horizontally with their load of alum, rise vertically to the upper floor, where they may feed any one of three 12-in. screw conveyors. These latter, each of which is 64 ft. in length, deliver the alum to the various bins through gated chutes. The screw conveyors are driven through friction clutches and bevel gears by a 7.5-h.p. motor at 1 200 rev. per min.

A dust collector system and filter is provided for the collection and filtration of the dusty air produced in handling the alum. A 5-h.p. motor at 1 200 rev. per min. operates the fan used for sucking the dust-laden air from the crusher, chutes, and rooms where it is produced, and for forcing it through the cloth filter.

The building is provided with a platform elevator, 5 ft. square, with a capacity of 2 500 lb. when moving at a speed of 150 ft. per min. A 13-h.p. motor at 900 rev. per min. is used for operating the elevator.

Electric current for operating the motors is 3-phase, 60-cycle, alternating current at 2 300 volts, which is reduced to 220 volts through three 15 kv-a transformers. A remote-control panel board with switches for operating the car plow, crusher, bucket carrier, screw conveyors, etc., is placed outside the room containing the power transformers. This same room also contains three 1.5 kv-a., 2 300 to 110-volt transformers for the lighting system. Electric current is supplied from generators in the Fairmount Pumping Station.

Chemical House.—The raw water entering this building through two 60-in. mains, passes through two Venturi tubes with throats 30 in. in diameter. The rate and total flow of water are shown on two meters, that indicate and record the rate of flow and also register the total flow. The rated maximum capacity of each meter is 120 000 000 gal. per day.

The bucket used to lift the alum from the storage bins, is filled while standing on a flat car, which runs on a narrow-gauge (24-in.) track beneath the bins. A turn-table enables the car with its load to be run into the hoist shaft leading to the top floor of the chemical house. The loaded bucket is lifted by a geared hoist (capacity 3 000 lb.) mounted on a mono-rail in the upper story of the chemical house. The hoist is operated by a 3-h.p. motor. When the bucket reaches the upper floors, it is moved by hand along the mono-rail to a scale (capacity 4 000 lb.), that forms a part of the monorail system. It is then transferred on the same rail system to a position directly over a circular opening in the cover of a dissolving tank. The bucket is lowered until it rests on this cover, when the bottom gate is opened, and the alum is discharged into the dissolving tank.

The electric current supplying power is brought into the building in the form of a 3-phase, 60-cycle, 2 300-volt alternating current, which is reduced to 220 volts through three 10 kv-a. transformers. Three 5 kv-a. transformers are also used to convert the 2 300-volt current to 110 volts for lighting purposes. The transformers are located in a separate room especially provided for this purpose.

The electric passenger elevator, in addition to being a convenience for the operating force, may serve as a means of carrying alum up from the bins to the dissolving tanks should the bucket hoist fail to operate. It has a lift of 56 ft., a capacity of 2 500 lb., and a maximum hoist speed of 150 ft.

per min., under full load. The shaft in which it operates is 6 ft. by 6 ft. 6 in. in plan. The elevator is driven by a 13-h.p. motor operating at 900 rev. per min.

The dust-collecting system consists of seven hoods, one over each possible loading position of the bucket. It also includes collector piping and valves and a dust arrester with a discharge hopper and outlet equipped with a dust valve and canvas spout. The fan is driven by a 10-h.p. electric motor running at 1 160 rev. per min.

The tanks for dissolving sulfate of alumina are each 6 ft. 6 in. by 5 ft. in plan and have a depth of 6 ft. 9 in. There is a 3-in. ledge 1 ft. above the bottom, which supports a wooden grid. This, in turn, supports a removable acid bronze screen upon which the alum is dumped. Below the bottom of the wooden grid, is a lead pipe distribution system for furnishing the dissolving water. In a recess at the end of the dissolving tank is a wooden baffle in line with the concrete wall below. Beyond that is an acid bronze screen, intended to remove trash of all kinds from the solution before it passes into the storage tanks. The wooden grid is made in three sections and is composed of $1\frac{5}{8}$-in. by $3\frac{5}{8}$-in. cypress joists set on edge with $\frac{7}{8}$-in. spacer blocks between the joists. The joists of each section are bolted together with $\frac{1}{2}$-in. bolts painted with an acid-proof paint before assembling. The bronze screens are set in cypress frames, and are composed of acid-proof bronze wire, 0.08 in. in diameter, 4 meshes per in. for the strainer screen and 6 meshes per in. for the screen resting on the wooden grid. These dissolving tanks were designed for a maximum charge of 4 000 lb. of chemicals.

Each solution storage tank is 15 ft. $4\frac{1}{2}$ in. by 18 ft. 3 in. in plan, with an average depth of approximately 10 ft. The bottom of the tank slopes 1 ft. 6 in. in its length, with a drain at the low point. The solution outlet from the tank is 19 in. above the drain. This design was adopted to form a sump into which trash or separated aluminum hydrate might settle and not be carried into the chemical pipe lines. To effect a mixing action in the solution tanks by driving water up through a grid at the bottom and supplying make-up water for bringing the solution to a predetermined strength, four $1\frac{1}{2}$-in. lead pipes spaced equi-distant across the bottom of the solution tank and tied into a 2-in. header are provided and are supplied with water through a 2-in. feed line. The four $1\frac{1}{2}$-in. laterals in the bottom of the tank are each provided with $\frac{1}{4}$-in. holes on 3-in. centers drilled in the top of the pipe.

Each orifice box is 1 ft. 9 in. by 4 ft. in plan and 3 ft. 9 in. deep, with the center line of the orifice set 1 ft. 11 in. above the bottom. Each box is divided into two compartments by a wooden baffle. Into one of the compartments the alum solution is discharged and passes over the baffle into the second compartment. The baffle acts to still the disturbance produced by the discharge of the solution. The float for maintaining a constant head over the orifice is placed in the second compartment. The orifice itself is placed in the wall of the box forming one end of this compartment.

The orifice boxes are designed in pairs with a common dividing wall. As the solution from either orifice leaves this box it passes into a common make-

up box, 18 in. wide, extending across the full width of the two orifice boxes. A 1-in. water line, discharging into this make-up box, permits an increase in the volume of solution which assists in better distribution as it is fed into the water.

The orifice itself has a fixed width of $\frac{3}{4}$ in. and is capable of being varied from a minimum length of $2\frac{1}{2}$ in. to a maximum of 7 in. The orifice plate is made of acid bronze and consists of a base plate set in the concrete to which the orifice plate proper is attached. One end of the orifice is designed to remain stationary while the other end is moved by a screw operated by a hand wheel. A pointer on the movable part of the plate indicates the opening on a scale set below.

For conveying the alum solution from the orifice boxes, a pipe leads from each set to a distributing trough over the rising well. This is a wooden trough, 70 ft. long, covered with sheet lead $\frac{3}{32}$ in. thick, and hung from bolts fastened to inserts placed in the concrete floor above.

The joints of the lead sheets are burned together, and on the down-stream side of the trough lead lips are spaced 2 ft. 6 in. apart. The trough is V-shaped, with a bottom angle of 90 degrees. It is 18 in. wide across the top and has a depth of 7 in. above the top of the fillet.

In order to obtain a uniform discharge of the solution from each lip, the level of the trough may be accurately adjusted by turnbuckles.

Administration and Filter Building.—There are four wash-water pumps on the floor at Elevation 216.0 for lifting the water from the effluent galleries to the wash-water tanks, in the top story of the Administration Building. They are operated through electric control actuated by means of floats in the wash-water tanks. The float contacts are arranged so that if the level of the water in the tanks continues to drop, additional pumps are thrown successively into service. The electric control panels are in the pump-room adjacent to the pumps. Push-button control is also provided in the pump-room. Two pumps, each with a capacity of 3 450 gal. per min., are driven by 75-h.p., induction motors of the squirrel-cage type, running at 1 760 rev. per min. Each of the other two pumps has a capacity of 1 850 gal. per min. and is driven by a 40-h.p. motor of the same type, running at a speed of 1 750 rev. per min. The current used in the motors for these pumps is a 3-phase, 60-cycle, at a potential of 2 300 volts.

Electric power is obtained from two sources for operating the motors on these pumps: From cables of the Municipal Electric Light Plant; and from the generators at the Fairmount Pumping Station.

The transformer and switchboard-room contains the four primary control panels for the four wash-water pumps; three 25-kv-a. transformers for converting the current from 2 300 volts to 220 volts, for general power purposes other than that required for the wash-water pumps; and three 25 kv-a. light transformers for changing the voltage from 2 300 volts to 110 volts for lighting purposes.

The large freight elevator (7 ft. by $6\frac{1}{2}$ ft.) serving all floors in the Administration Building, has a capacity of 10 000 lb. at a speed 75 ft. per min. The power is obtained from a 27-h.p. electric motor, operating at 900 rev.

per min. The machine shop is wired for machine tools, and a switchboard control panel has been installed. The machine tools themselves have not as yet been purchased.

In the foremen's room on the main floor of the Administration Building is located the wash-water Venturi meter register. The Venturi tube is 24 in. in diameter, with a 14-in. throat.

In this same room are two indicating depth gauges, one for each basin of the Baldwin Reservoir, and one pressure gauge to indicate the depth of water in the wash-water tanks. A high-water electric alarm bell notifies the operator when too high a level is reached in the wash-water tanks.

The chlorine room on the floor at Elevation 234.0 is located with one wall just beyond the outlet end of the filtered water conduit at Elevation 216.0. A pit 1 ft. 6 in. by 3 ft. 6 in. and 2 ft. 3 in. deep is built into the floor of this room. Passing through the wall of the pit are three $2\frac{1}{2}$-in. galvanized-iron pipes, 7 ft. 6 in. long, and extending from 6 in. inside the pit wall to a point over the center line of the outlet end of the filtered water conduit. A manhole is located in the floor at Elevation 234.0, just over the end of these pipes. A heavy rubber hose fed by the chlorine machines may be placed in each pipe with its outlet end submerged in the water of the effluent channel leading to the clear-water reservoir.

For feeding the chlorine, there are two control units each with a maximum capacity of 320 lb. per day. The room in which these machines are located, is equipped with two vestibule entrances, each having two sets of double doors to prevent the spreading of fumes to other parts of the building. A ventilating system is also provided for exhausting the fumes from the chlorine room proper. The exhaust fan is located on the wash-water tank-room floor, and is driven by a 5-h.p. motor operating at 900 rev. per min.

An operating table is provided for each filter tank (Fig. 18). It consists of an ornamental cast bronze frame with statuary finish and with Tennessee marble panels and top. On the top are four levers equipped with indicators. These levers are used for opening and closing four-way cocks which admit and release water under pressure to the hydraulic cylinders used to open and shut the influent, effluent, sewer, and wash-water valves. A shut-off valve for the pressure manifold, and bronze casings for holding the loss of head and rate-of-flow gauge mechanisms, are also placed on the top of the table. In the hood of each gauge is a small electric bulb for illuminating the face of the gauge at night. The marble panels at the back of the table are omitted, the opening being closed, however, by a brass wire screen for ventilating purposes. The operating tables and fixtures were especially designed for this plant.

The 8-in. pressure supply line for operating the hydraulic valves is under a pressure of 150 lb. per sq. in. As this is too high for this purpose, it is reduced by an 8-in. pressure-regulating valve to about 110 lb. per sq. in. All waste lines from the four-way cocks discharge into the main gutter of the filter.

The effluent valve of each filter is placed down stream from the rate controller and this permits each controller to be emptied for inspection or re-

pair without draining the effluent galleries. This arrangement, however, subjects the controller and connected mercury pots to the full head of the wash-water pressure. This necessitates the cutting off of wash-water pressures, during the washing process, from the mercury pots for the rate of flow and loss of head gauges. The effluent valve, therefore, when closing, automatically closes two small hydraulic valves in the small pressure pipe lines leading from the controller to the mercury pots. These devices prevent the mercury from being forced out of the pots while the controller is under pressure in the washing process. These cut-off valves open automatically when the effluent valve is again opened.

Lighting.—A very complete system of lighting has been installed in all the buildings. Special lighting for the filter tanks enables operators to view the filtering and washing processes at night. The offices and laboratories are extremely well lighted. Ornamental fixtures and lights are installed in the end and cross pavilions of the wings of the filter building.

Heating.—A low-pressure (5-lb. gauge and 10-in. vacuum) vacuum type of steam heating is provided for all the buildings except the gate-houses. Thermostatic control is used in the Administration Building. Steam is supplied from the boiler plant at the Fairmount Pumping Station.

Telephone System.—A 24-station intercommunicating telephone system furnishes contact between the offices, laboratories, foremen's rooms, and other important points in the plant. Duplicate sets of talking and ringing batteries are located in the machine shop, and are kept constantly charged by a motor generator set.

Completion of Baldwin Filtration Plant

The construction of the Baldwin Filtration Plant and Reservoir, as well as of all allied projects necessary to make the operation of this plant possible, was hastened in every way, in order to derive the benefit of increased pressures in the low-service district due to the use of the new reservoir.

All the various projects had been so nearly completed, that in June, 1925, it was possible to test and clean out the regular water channels in the Baldwin Filtration Plant from the chemical house through the filtered-water by-pass in the Administration Building, to Baldwin Reservoir. On June 29, 1925, raw chlorinated water was first passed through the previously mentioned water channels into Baldwin Reservoir and on July 1, 1925, this reservoir was connected to the low-service distribution system.

From the average results of twenty-two gauging stations, throughout the low-service district, the increased pressure due to the change to the higher reservoir was 16.6 lb. This was the difference between the average of the readings at 9:00 A. M., before cutting in the new reservoir, and that at 2:30 P. M., after the system had been changed over. This increase in pressure was obtained with the water in Baldwin Reservoir about 7½ ft. below the overflow.

The placing of the filter equipment progressed so satisfactorily that on September 15, 1925, the filtered-water by-pass was closed and the first chemi-

cally treated water was passed through the coagulation basins and to the west twenty filters, thus placing one-half the plant in service. On October 20, the remaining filters were available for service.

The total elapsed time required for the construction of the plant was slightly more than three years and eleven months. This is computed from the date of awarding the contract under which the reservoir was constructed, to the date on which all the filters were ready for service, and the plant was ready for output of its full capacity. It excludes the time required for the excavation of the reservoir and coagulation basins.

Amounts of Contracts and Analysis of Costs

The various items entering into the cost of Baldwin Filtration Plant are given in Tables 2 and 3. In preparing this detailed statement, the actual cost of the asset is given; this cost is also adjusted to a base cost in accordance with the purchasing value of the 1913 dollar. This was done in order to permit cost comparisons on the basis of pre-war values. The cost index figures as prepared by the *Engineering News-Record,* were used in making this adjustment. A comparison of unit costs may readily be made by means of Table 2. For example, the alum storage building, with a capacity of 1 000 tons, cost $132 per ton for storage. On the 1913 basis, the cost is $63 per ton of storage. Similarly, the four coagulation basins, with a total capacity of 32 800 000 gal., cost $49 425 per 1 000 000 gal. The capacity of Baldwin Reservoir is 135 000 000 gal. and its cost is seen to be $21 458 for each 1 000 000 gal. of storage.

Table 3 gives a summary of costs of the units comprising the filter plant proper, including both actual and 1913 values. It also gives for each of these values the cost of each unit per 1 000 000 gal. daily of capacity. This table excludes from consideration, the following items: (1) Mains delivering raw water to the rising well at the chemical house; (2) mains that take water away from outlet Gate-House No. 7; (3) changes at Fairmount Reservoir; (4) landscape development; (5) cost of all land; and (6) rough grading of the filter plant grounds, engineering, inspection, and overhead.

The large capacity of the Baldwin Reservoir is due to the fact that it is used as a storage reservoir for the low-service district. As the normal capacity of a filtered-water reservoir for a filtration plant is about 25% of the normal daily rated capacity of the plant, it is not proper to charge the entire cost of the Baldwin Reservoir to the filtration plant. Accordingly, while Table 3 includes costs for the plant as actually constructed, data are also included for the purpose of comparison, giving costs on the basis of the customary size of a filtered-water reservoir.

A capacity of 40 000 000 gal. would be sufficient for a plant of this size. The exact cost of this smaller reservoir could only be obtained by receiving bids based on an actual design, but for the purpose of comparison it will be assumed that the cost of the smaller, theoretical reservoir would be $\frac{40}{135}$ of the cost of the reservoir as actually built. The totals of Items A, B, C, D, and F, as given in Table 3, represent these adjusted values.

TABLE 2.—DETAILED COSTS OF BALDWIN FILTRATION PLANT.

Units constructed.	Actual cost of asset.	Cost reduced to 1913 basis by use of cost index figures.	Date of receipt of bids.	Cost index based on the year 1913 as 100.
Control Works at Fairmount Reservoir: Substructure:				
Contract 18, Foundations of four gate-houses, shafts, branch and main tunnels to header at pumping station, distributing pipe into reservoir, and all sluice-gates................	$ 209 891.76	$ 109 318.63	January 12, 1923	192
Contract 41, Guniting areas, concrete floor of reservoir, and slope wall paving around new openings made in floor and slope walls......	4 548.78	2 125.60	April 13, 1923	214
Total, substructure........................	$ 214 440.54	$ 111 444.23	....	...
Control Works at Fairmount Reservoir: Superstructure:				
Contract 42, Superstructures of four gate-houses..	$ 53 243.00	$ 24 201.36	October 24, 1923	220
Alum Storage Building (Foundation Area, 0.12 acres):				
Contract 57, Building complete, including foundations, superstructure, and alum-crushing and conveying equipment.				
Total..	$ 131 972.64	$ 62 844.11	January 2, 1925	210
Chemical House and Mixing Flume: Substructure and Equipment (Foundation Area, 0.65 acres):				
Contract 51-S, Substructure complete, including excavation and back-fill, concrete foundation and reinforced concrete skeleton of building, mixing flume, and chemical house equipment, including concrete dissolving, solution, and orifice tanks, piping and valves for chemical solution and alum-handling equipment..........................	$ 378 407.54	$ 177 656.12	August 20, 1924	213
Contract 44, Additional work.............	493.50	227.42	May 25, 1923	217
Total, substructure and equipment.....	$ 378 901.04	$ 177 883.54	....	...
Chemical House and Mixing Flume: Superstructure:				
Contract 54-S, Superstructure complete, including all brick and cut-stone work, steel roof trusses, slate roof, partitions, sash, doors, painting, etc..............................	$ 215 000.00	$ 100 938.97	....	...
Elevator..	6 500.00	3 051.64	....	...
Electric wiring and lighting fixtures..........	16 500.00	7 746.48	....	...
Heating and ventilating.......................	6 000.00	2 816.90	....	...
Plumbing complete............................	5 000.00	2 347.42	....	...
Total, superstructure....................	$ 249 000.00	$ 116 901.41	August 20, 1924	213

TABLE 2.—(*Continued*).

Units constructed.	Actual cost of asset.	Cost reduced to 1913 basis by use of cost index figures.	Date of receipt of bids.	Cost index based on the year 1913 as 100.
Coagulation Basins: Substructure (Foundation Area, 7.74 acres):				
Contract 1, Excavation	$ 162 069.63	$ 61 158.35	April 20, 1920	265
" 2, Foundation under west conduit	33 521.82	20 193.87	November 4, 1921	166
" 44, Completion of excavation and constructing completely the concrete cover basins, including drains, proportionate cost of outlet end of main drain, foundations, walls, conduits, roof, roof back-fill, sluice-gates, mud-valves and pressure-pipe cleaning system	1 425 556.64	656 938.54	May 25, 1923	217
Total, substructure	$1 621 148.09	$ 738 290.76	....	...
Coagulation Basins: Superstructure:				
Contract 47, Superstructures of six gate-houses and brick and stone facing of west wall of basins	$ 106 000.00	$ 48 181.82	October 24, 1923	220
Filters and Administration Building: Substructure (Foundation Area, 2.48 acres):				
Contract 1, Excavation	$ 68 394.51	$ 25 809.25	April 20, 1920	265
" 44, Proportionate cost of outlet end of main drain	25 000.00	11 520.73	May 25, 1923	217
Contract 47, Completion of excavation, building drains, construction of all foundations, concrete effluent galleries, filter tanks, and conduits, placing all valves in pipe gallery and furnishing and placing all pipe gallery and effluent gallery piping	1 524 195.61	692 816.19	October 24, 1923	220
Contract 49-K, Furnishing hydraulically-operated filter valves	85 786.50	38 642.57	September 14, 1923	222
Contract 49-R, Furnishing hand-operated valves	15 109.40	6 806.04	September 14, 1923	222
Total, substructure	$1 718 486.02	$ 775 594.78	....	...
Filter and Administration Building: Superstructure:				
Contract 47, Superstructure complete above the tops of the filter tanks, including building skeleton and roof, all brick cut-stone work, metal lath, plaster, painting, etc	$1 250 530.00	$ 568 422.73	October 24, 1923	220
Contract 48-E, Electrical wiring and fixtures	31 000.00	14 090.91	October 24, 1923	220
" 48-S, Heating and ventilating system	34 659.00	15 754.09	October 24, 1923	220
" 48-W, All plumbing	29 775.05	13 534.11	October 24, 1923	220
" 54-B, Elevator	7 260.00	3 392.52	July 23, 1924	214
Total, superstructure	$1 353 224.05	$ 615 194.36	....	...

TABLE 2.—(*Continued*).

Units constructed.	Actual cost of asset.	Cost reduced to 1913 basis by use of cost index, figures.	Date of receipt of bids.	Cost index based on the year 1913 as 100.
Filter Equipment:				
Contract 54-B, Furnishing and placing filter strainer system; furnishing and placing filter rate controllers and operating tables; furnishing and placing electrically driven wash-water pumps, transformers, and pump-motor control; furnishing and placing chlorine feeding apparatus; furnishing and placing office furniture, floor covering, and stock room shelving; and placing filter gravel and sand	$ 243 680.00	$ 118 869.15	July 23, 1924	214
Contract 52-C, Furnishing filter sand..........	33 550.00	16 286.41	November 6, 1924	206
Contract 52-O, Furnishing filter gravel..........	18 237.25	8 853.03	November 6, 1924	206
Contract 47, Steel wash-water tanks..........	13 500.00	6 186.36	October 24, 1923	220
Contract 47, Metal shelving, cupboards, and lockers..........	3 300.00	1 500.00	October 24, 1923	220
Laboratory work tables, cabinets, and benches..........	12 499.35	6 097.24	October 30, 1925	205
Total, filter equipment..........	$ 324 766.60	$ 152 742.19	....	...
Baldwin Reservoir: Substructure (Foundation Area, 13.81 acres):				
Soil stripping by direct labor in 1915..........	$ 26 386.00	$ 28 372.04	1915	93
Excavation contract..........	183 660.40	204 067.11	April 2, 1915	90
Excavation by direct labor..........	809 334.00	408 754.55	1919	198
Contract 2, Construction of concrete-covered reservoir complete, including completion of excavation, floor and wall drains, foundations, walls, roof, back-fill on roof, sluice-gates and all valves and pipe within Gate-House No. 7..........	1 873 735.36	1 128 756.24	November 4, 1921	166
Contract 47, Additional work..........	3 755 00	1 706.82	October 24, 1923	220
Total, substructure..........	$2 896 870.76	$1 771 656.76	....	...
Baldwin Reservoir: Superstructure:				
Contract 47, Superstructure of Gate-House No. 7 complete..........	$ 80 000.00	$ 36 363.64	October 24, 1923	220
Rough Grading of Filtration Plant Grounds: (Area of grounds, exclusive of area occupied by structures, 28.99 acres):				
Contract 1, Excavation..........	69 883.70	26 371.21	April 20, 1920	265
" 2, Excavation and fill..........	50 838.00	30 625.30	November 4, 1921	166
" 44. Fill..........	16 957.60	7 814.56	May 25, 1923	217
" 54-S, Fill..........	5 420.00	2 544.60	August 20, 1924	213
Total..........	$ 143 099.30	$ 67 355.67	....	...
Miscellaneous:				
Steam main to Plant A (July, 1925)..........	$ 25 200.00	$ 12 292.68	July, 1925	205
Steam main to Plant B (September, 1925)..........	9 720.00	4 811.88	September 25, 1925	202
Dump truck for hauling alum (October, 1925)..........	4 310.55	2 102.71	October 16, 1925	205

TABLE 3.—ACTUAL COST OF BALDWIN FILTRATION PLANT.

Item.	Units constructed.	Sub-structure.	Super-structure.	Filter equip-ment.	Total.
	SUMMARY OF TOTAL COSTS.				
A	Alum Storage Building	$ 131 972.64*	....	....	$ 131 972.64
B	Chemical House and Mixing Flume.....	378 901.04	$ 249 000.00	....	627 901.04
C	Coagulation Basins......................	1 621 148.09	106 000.00	....	1 727 148.09
D	Filters and Administration Buildings...	1 718 486.02	1 353 224.05	$324 766.60	3 396 476.67
E	Baldwin Reservoir........................	2 896 870.76	80 000.00	....	2 976 870.76
	Total..............................	$6 747 378.55	$1 788 224.05	$324 766.60	$8 860 369.20
F	Estimated cost of a 40 000 000-gal. reservoir $(=\frac{40}{185}\times \text{Item E})$	$858 332.08	$23 703.70	....	$882 035.78
	Total of Items A, B, C, D, and F..	$4 708 839.87	$1 731 927.75	$324 766.60	$6 765 534.22
	TOTAL COSTS REDUCED TO COST PER MILLION GALLONS DAILY CAPACITY OF PLANT.				
A	Alum Storage Building....................	$ 799.83*	....	....	$ 799.83
B	Chemical House and Mixing Flume.....	2 296.37	$ 1 509.09	....	3 805.46
C	Coagulation Basins......................	9 825.14	642.42	....	10 467.56
D	Filters and Administration Buildings...	10 415.07	8 201.36	$1 968.28	20 584.71
E	Baldwin Reservoir........................	17 556.79	484.85	....	18 041.64
	Total..............................	$40 893.20	$10 837.72	$1 968.28	$53 699.20
F	Estimated cost of a 40 000 000-gal. reservoir $\left(=\frac{40}{185}\times \text{Item E}\right)$..............	$5 202.01	$143.66	....	$5 345.67
	Total of Items A, B, C, D, and F....	$28 538.42	$10 496.53	$1 968.28	$41 003.23
	SUMMARY OF TOTALS ADJUSTED BY COST INDEX FIGURES TO CONFORM TO THE 1913 BASIS.				
A	Alum Storage Building....................	$ 62 844.11*	....	....	$ 62 844.11
B	Chemical House and Mixing Flume.....	177 883.54	$116 901.41	....	294 784.95
C	Coagulation Basins......................	738 290.76	48 181.82	....	786 472.58
D	Filters and Administration Buildings...	775 594.78	615 194.36	$152 742.19	1 543 531.33
E	Baldwin Reservoir........................	1 771 656.76	36 363.64	... .	1 808 020.40
	Total..............................	$3 526 269.95	$816 641.23	$152 742.19	$4 495 653.37
F	Estimated cost of a 40 000 000-gal. reservoir $\left(=\frac{40}{185}\times \text{Item E}\right)$................	$524 935.34	$10 774.41	....	$535 709.75
	Total of Items A, B, C, D, and F....	$2 279 548.53	$791 052.00	$152 742.19	$3 223 342.72

* Includes superstructure.

TABLE 3.—(*Continued.*)

Item.	Units constructed.	Sub-structure.	Super-structure.	Filter equip-ment.	Total.
	TOTAL ADJUSTED COSTS REDUCED TO COST PER MILLION GALLONS DAILY CAPACITY OF PLANT.				
A	Alum Storage Building	$ 380.87*	....	....	$ 380.87
B	Chemical House and Mixing Flume	1 078.08	$ 708.49	....	1 786.57
C	Coagulation Basins	4 474.49	292.01	....	4 766.50
D	Filters and Administration Buildings	4 700.57	3 728.45	$925.71	9 354.73
E	Baldwin Reservoir	10 737.31	220.39	....	10 957.70
	Total	$21 371.32	$4 949.34	$925.71	$27 246 37
F	Estimated cost of a 40 000 000-gal. reservoir $\left(= \frac{40}{135} \times \text{Item E}\right)$	$3 181.43	$65.30	....	$3 246.73
	Total of Items A, B, C, D, and F	$13 815.44	$4 794.25	$925.71	$19 535.40

* Includes superstructure.

ACKNOWLEDGMENT

As this project has covered a considerable period of time, the work was carried on under four different City Administrations and great credit is due to the City's officials and Councils for their continued support in carrying this project to a successful completion.

The work has been conducted under the general supervision of the Directors of Public Utilities, Messrs. C. W. Stage, Thomas S. Farrell, A. B. Roberts, E. L. Myers, and Howell Wright, and Commissioners of Water, Messrs. C. F. Shultz, J. T. Martin, A. V. Ruggles, and C. P. Jaeger. During the entire development and construction of this project J. W. Ellms M. Am. Soc. C. E., was Engineer of Water Purification and Messrs. A. V. Ruggles and A. G. Levy, Members, Am. Soc. C. E., were Engineers of Construction and Survey. The Frazier-Sheal Company with J. E. A. Linders, M. Am. Soc. C. E., as Engineer of Design, and H. T. Hammer as Resident Engineer, had charge of the design and construction of Baldwin Reservoir. The design and construction of the remaining part of the project was handled directly by the Engineering Department of the Division of Water with Messrs. A. G. Levy and G. W. Hamlin, M. Am. Soc. C. E., as Engineers of Design and the latter as Resident Engineer. Herman Kregelius, City Architect, designed all the superstructures in connection with this project.

The plant was completed and put into operation under the administration of City Manager W. R. Hopkins and Director of Public Utilities Howell Wright, to whose energy and ability the early completion of the project was largely due.

CONCLUSIONS

The project involved an expenditure of approximately $10 000 000. and contains engineering features which should be of interest to all those en-

gaged in the design and construction of works of this type. The descriptions are in considerable detail in order to enable other engineers to grasp the essentials of the design and construction and to indicate the magnitude of the work. Certain features like the hydraulic-jump mixing flumes are a departure from the old methods of mixing chemical solutions with the water to be treated; the design of the coagulation basins in providing a straight flow through them with no baffles is an improvement which has proved entirely satisfactory in practice and which is a departure from existing methods; and the use of the sharply sloping floor in the coagulation basins for the dual purpose of reducing the height of the outside walls and facilitating the cleaning of the basins is of practical importance.

The design and construction of Baldwin Reservoir on account of its size, should be of considerable interest to engineers, since it is the largest covered concrete reservoir that was ever constructed. The methods of introducing the water to the reservoir and drawing it off in order to provide good circulation is a point of interest. The groined arches used for the roof have the longest span constructed to date.

The very complete tables of costs are so rarely found in engineering literature that the figures given should be particularly valuable since they represent the careful analysis of all the contracts involved in the project.

The plant is adjacent to the Park System of the city and a proper architectural treatment was essential. The buildings present a most imposing appearance and the beauty of the design as a whole has been rarely, if ever, equalled.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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LAMINATED ARCH DAMS WITH FORKED ABUTMENTS

By Fred A. Noetzli,* M. Am. Soc. C. E.

Synopsis

This paper describes some modifications of arch dam form and layout, as compared with usual current practice, which are believed to promote both simplicity of design and economy. Simplicity of design is achieved by the use of structural proportions that avoid the complexities of the "thick arch", and economy, fortunately, follows in consequence.

That the controlling principles hereafter noted, and the ends to be sought, are now recognized in some quarters, is evidenced by several recent dam designs, but it is believed that this conception may advantageously be carried further than it was in these instances. The purpose of this paper is to present the orderly development and application of certain basic principles, carried to their logical conclusion.

It is apparent that the efficient concrete arch is one in which the stresses are approximately equal across any section; that is, in which the line of thrust approximately coincides with the neutral axis. This condition is most nearly obtainable in a dam composed of thin arches; it is impossible to secure in "thick arches." Furthermore, as arches depart from the "thin" classification, and approach the "thick", the relation between the radius, the thickness, and the central angle, has important consequences upon the stress distribution; and, other things being equal, the maximum angle will generally give minimum stress. With these considerations in view, a typical structural form and layout has been developed by the writer, such as will permit their attainment to the maximum practical degree. The topography at the dam site is obviously a controlling feature but, for typical conditions, the desired results are obtained by:

1.—The use of a special type of buttress to take the thrust of the upper arch elements, thereby reducing the required maximum arch span, and hence the maximum radius.

Note.—Written discussion on this paper will be closed in August, 1930, *Proceedings*.

* Cons. Hydr. Engr., Los Angeles, Calif.

2.—The pronounced sloping, up stream from the crest, of the crown of the dam, thereby securing the minimum radii for the lower arch elements.

3.—By the subdivision, where indicated, of the total arch thickness into two or more separate arches, or arch laminæ, thereby keeping the arches "thin", or relatively so, and maintaining throughout the dam the corresponding advantages.

Principle 1 requires the use of gravity wings which are so related to the buttresses as greatly to increase the strength of the latter, and to provide ample assurance against over-stress in this quarter. Principles 1 and 2 have been applied by the writer to the design and construction of a dam hereafter described. The method of lamination has been adopted in principle by French engineers on a large dam now under construction. It is believed that the method proposed by the writer is equally as effective and possesses some advantages over the French design.

Introduction

The first arch dams were designed by the simple cylinder formula. Most of these structures are of moderate span and height, and the unit stresses assumed in the design were relatively low. No arch dam designed by this method has failed for reasons of structural weakness. This speaks rather well for the adaptability of such structures to the loads and stresses caused by the water pressures.

A great improvement in the design of arch dams accompanied the application of the elastic theory. Few other branches of structural engineering have received, during the past ten years, as much attention as dams of the arch and multiple-arch type. This is evidenced by the large number of theoretical studies published during this period in the United States and in foreign countries.* Tests and experiments were made on a number of dams and on laboratory models, to determine within reasonably close limits the laws of the distribution of stresses in arch dams. The most signal advance in this connection was the construction and successful testing of the Stevenson Creek Experimental Arch Dam by the Committee of Engineering Foundation on Arch Dam Investigation. Much valuable and useful information has been obtained from these theoretical and experimental studies.

Theoretical investigations have shown that, for thick arches of dams, certain assumptions ordinarily made in the analysis of relatively thin arches—as used in arch bridges and arch dams of moderate height and span—could be improved upon, and the theory of thick arches was developed. However, apparently it was found that this theory has a rather limited field of application, since it often becomes necessary to abandon the actual arch as a structural element, and have recourse to a contained "secondary arch".†

This "secondary arch" is assumed to support the entire water load, and is analyzed accordingly. The basic assumption related to these secondary arches,

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* See Bibliography in Report on Arch Dam Investigation, *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, p. 277.

† *Transactions*, Am. Soc. C. E., Vol. 90 (1927), p. 475.

inside the primary thick arch, is that numerous cracks are formed in the tension zone of the latter; that is, on the extrados at the abutments and on the intrados at the crown. In other words, the most advanced theory serves to show that, in some instances, the structure cannot function as built, and in such cases it is customary to neglect the concrete that is presumably cracked or in tension. The conditions which determine the secondary arches are very complex, and involve some rather uncertain assumptions; but even if this solution were clean cut and definite, it would still be open to the objection that to place deliberately in a dam concrete which it is assumed will be structurally inoperative, is not in accord with efficient design or engineering.

In his early studies of arch dams the writer was impressed with the difficulties involved in a theoretically exact analysis. His further studies, and the knowledge derived from the published work of others, have served to strengthen his original impression. It appears that each advance made in the development of the structural theory has not only introduced additional mathematical complications, but necessarily has involved fundamental assumptions which are admittedly uncertain of determination. In other words, despite the great advance that has been made in mathematical analysis of arched dams in recent years, the inherent complexities and uncertainties of the problem are still present.

When a structural type is such that its analysis is complicated and uncertain beyond a reasonable degree, and when it is evident that primary structural elements cannot function as built, a modification of type, such as to result in simple relationships, is perhaps indicated. Great complexity of design can only be justified by producing corresponding efficiency, which is questionable in the case of the present thick arch dams.

Some of the major difficulties encountered in the analysis of the current type of arched dam are a result of the use of thick and flat arches. The consequent complications introduced are twofold: First, as regards the action of the arches themselves, and, second, the action of the stiff cantilever elements, and their influence upon the arches. It is known that the action of a thin arch under water loads, as in the upper part of a thin arched dam, is relatively simple and determinate. It may also be readily shown that, regarding arching action only, the thin arch is usually far more efficient than the thick. So far as the arches themselves are concerned then, it would obviously be desirable to keep them as thin as possible, within practical limits. As the thickness of the arches is reduced, the flexibility of the vertical or cantilever elements is increased in very much greater proportion, resulting in a greatly reduced influence of these elements, and an increase in the proportion of load carried by the arches.

On the other hand, an increase in the efficiency of the cantilever elements (by increasing their thickness) can only be accomplished at the expense of reduction of arch efficiency. In other words, it is assumed that the thin and efficient arch may be taken as the goal of attainment, without any material sacrifice of other benefits.

With the preceding considerations in view, the writer for some time past has devoted his efforts toward the development of a type of arch dam which

would eliminate or reduce some of the difficulties and complexities of the current thick arch type, and, at the same time, give a structure of greater efficiency. The purpose of this paper is to describe the type of structure evolved and the manner in which the desired ends are accomplished.

In what has preceded, it was stated that the thin arch, subjected to water loads as in a dam, possessed marked advantages over the thick arch under similar conditions. This fact is probably well understood, but it will be pertinent to summarize here these relations in greater detail.

As has been shown by recent investigations, the influence of rib-shortening, temperature, and shrinkage becomes very marked as the arch thickness is increased. Their effect is to create an eccentricity of the resultant thrust, and, therefore, an unequal distribution of pressure across the section, tending toward or developing tension at the crown intrados and at the extrados at the abutments. The fact that these important influences are neglected in the cylinder formula is one of the main reasons that this is not applicable to thick arches.

In all, or most all, of the theoretical investigations of the arched dam, the analysis has been based upon horizontal elementary arches, consequently subjected to uniform normal water pressure. That is, these are assumed to be the primary elements, or those carrying the maximum stress. Depending upon the layout of the dam, and other considerations, the actual primary elements may be horizontal or inclined, but it is probable that conclusions pertaining to horizontal elements will be applicable in kind to inclined elements, and will serve as a measure of the conditions obtaining in the latter. The complexities introduced by a consideration of inclined elements would prohibit any generalized theory or conclusions, in the interests of which it is permissible to adopt the horizontal arch element as the argument, realizing that the results thus obtained may be subject to subsequent modification. In this, the writer follows his predecessors.

Notation

The following notation is used in the paper:

t = thickness of arch, in feet.
r = mean radius of arch, in feet.
R_u = radius of extrados, in feet.
R_a = radius of intrados, in feet.
y = ordinate of any point on the arch axis.
ϕ = angle described by any radial line moving from the crown toward the abutment.
ϕ_1 = one-half the central angle, in degrees.
σ_c = cylinder stress, in pounds per square inch.
σ_r = rib-shortening stress, in pounds per square inch.
σ_T = stress corresponding to a drop in temperature.
H_T = normal pull due to a drop of temperature per foot width of arch, at the crown.
H_r = normal rib-shortening at the crown per foot width of arch.
p' = normal radial pressure, in pounds per square foot, on extrados.
T = temperature drop in degrees Fahrenheit.
n = number of laminae in the arch dam under consideration.
e = coefficient of thermal expansion.

E = modulus of elasticity of arch, in pounds per square inch.
C = coefficient determined from Figs. 1 and 2 (see Equation (5)).
K = coefficient determined from Figs. 3 and 4 (see Equation (7)).

Analysis of Stresses in Circular Arches by Means of Charts

From formulas and diagrams* by William Cain, M. Am. Soc. C. E., it is shown that, other things being equal, the rib-shortening and temperature stresses are determined by the central angle, $2\ \phi_1$, and the ratio, $\frac{t}{r}$, of the arch thickness to the mean radius. It is evidently desirable to use such values of these quantities, in so far as may be practicable, as will give minimum values of these stresses, and, hence, maximum efficiency of the arch.

The normal rib-shortening pull at the crown, per foot width of arch, is given by the equation,†

$$H_r = -\frac{K'}{12}\, p'\, R_u \left(\frac{t}{r}\right)^2 \qquad (1)$$

wherein, $K' = \frac{2 \sin \phi_1}{D_s}$.

Likewise, the equation for cylinder stress is,

$$\sigma_c = \frac{p'\, R_u}{144\ t} \qquad (2)$$

Transposing algebraically to determine a value of $p'\, R_u$ and substituting this value in Equation (1),

$$H_r = -12\ K'\ \sigma_c\ t \left(\frac{t}{r}\right)^2 \qquad (3)$$

The rib-shortening stress at any section may be expressed by,

$$\sigma_r = -\frac{H_r \cos \phi}{144\ t} \pm \frac{H_r\, y\, r}{24\ t^2} \qquad (4)$$

Substituting the value of H_r from Equation (3), combining and reducing, Equation (4) becomes,

$$\sigma_r = \sigma_c \frac{K'}{12} \left(\frac{t}{r}\right)^2 \left[-\cos \phi \pm 6\ y \frac{r}{t} \right] = C\ \sigma_c \qquad (5)$$

thus giving the rib-shortening stress in terms of the cylinder stress. For a drop in temperature, the normal pull at the crown per foot width of arch is given by the equation,‡

$$H_T = -\frac{K'}{12}\ E\ e\ T\ t \left(\frac{t}{r}\right)^2 \qquad (6)$$

In this equation, T is considered positive, E is in pounds per square foot, and t is in feet. After reducing E to pounds per square inch this expression is

* "The Circular Arch Under Normal Loads," *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922). p. 233; see, also, "Design and Construction of Dams," by Edward Wegmann, Eighth Edition, 1927, chapter on "Multiple-Arch Dams."

† "Design and Construction of Dams," by Edward Wegmann, M. Am. Soc. C. E., Eighth Edition, 1927, Equation (9), p. 445.

‡ *Loc. cit.*, Equation (14), p. 448.

identical with Equation (3), except that σ_c is replaced by $E\,e\,T$. The equation for temperature stress at any section may be developed to read,

$$\sigma_T = E\,e.\,T\,\frac{K'}{12}\left(\frac{t}{r}\right)^2\left[-\cos\phi \pm 6\,y\,\frac{r}{t}\right] = C\,E\,e\,T \quad\ldots\ldots(7)$$

in which, C is the same as in Equation (5). In other words, after certain simple quantities have been computed, the same coefficient, C, is applicable to determine both the rib-shortening and temperature stresses.

Values of C corresponding to extrados and intrados at both crown and abutment have been computed and plotted on Figs. 1 and 2. It is believed that these curves present the required values in such form as to give a clear picture of the influence of the variables, over a wide range.

It will be observed that when $\frac{t}{r} = 0$, the value of C is zero for all values of $2\,\phi_1$. In other words, as $\frac{t}{r}$ approaches zero, the stress approaches the simple cylinder stress. It is interesting to note also that (within the range of the curves) except for very large angles, the coefficient, C, and, hence, the stress, increases as $\frac{t}{r}$ increases from zero up to some point, and then decreases with further increase in $\frac{t}{r}$. In some cases, and for the smaller angles, the maximum value of the stress corresponds to relatively small values of $\frac{t}{r}$. Also, for some values of $\frac{t}{r}$, the stress may be smaller for smaller angles. Both these facts are perhaps contrary to what would be expected. In other words, the curves show that not for all conditions the rib-shortening and temperature stresses are decreased by decreasing the arch thickness and increasing the central angle.

The sign of the coefficient, C, determines the sign of the rib-shortening stress, σ_r, and also that of the temperature stress, σ_T, for a drop in temperature, which generally is the only direction that need be considered. The total stress due to load is evidently equal to,

$$\sigma_c + \sigma_r = \sigma = (1 + C)\,\sigma_c \quad\ldots\ldots\ldots\ldots\ldots\ldots(8)$$

Obviously, there is tension on the section when C is negatively greater than $-$ 1.0, so that the horizontal representing this value on the charts defines the limiting conditions for absence of tension due to load forces.

From the standpoint of design, it may sometimes be advisable to fix σ and to secure such values of $\frac{t}{r}$ and $2\,\phi_1$ as will accord with this. To satisfy this condition, the value of σ_c must equal $\frac{\sigma}{(1 + C)}$, which must be obtained by trial.

An inspection of the curves will show that the maximum compressive stress will occur at the abutment intrados (Fig. 2(b)), and the minimum compression or maximum tension at the abutment extrados. Fig. 2(a) shows that for even moderately large angles and fairly small values of $\frac{t}{r}$, the tensile

stresses, due to load only, may become very large. These would be further increased, of course, by the temperature stresses, so that the combined stress may readily become critical.

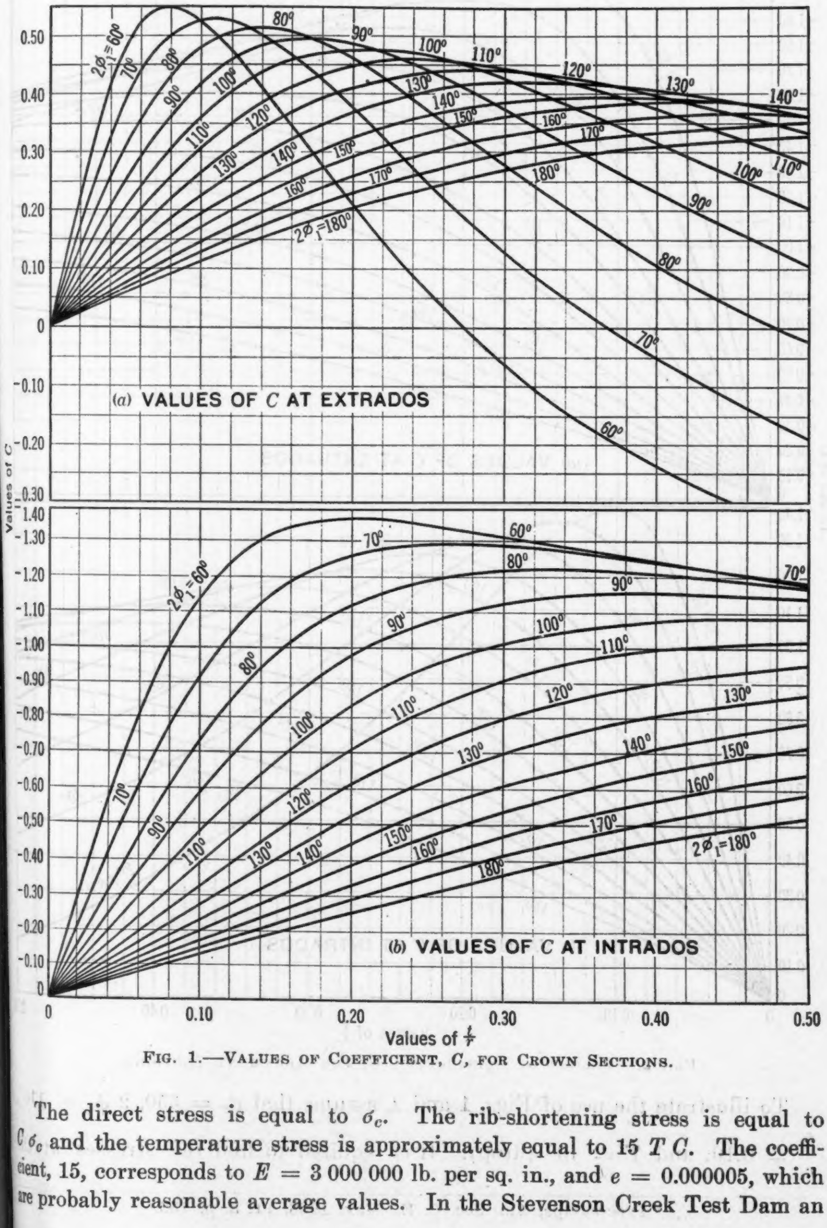


FIG. 1.—VALUES OF COEFFICIENT, C, FOR CROWN SECTIONS.

The direct stress is equal to σ_c. The rib-shortening stress is equal to $C\,\sigma_c$ and the temperature stress is approximately equal to 15 $T\,C$. The coefficient, 15, corresponds to $E = 3\,000\,000$ lb. per sq. in., and $e = 0.000005$, which are probably reasonable average values. In the Stevenson Creek Test Dam an

average value of $E = 3\,600\,000$ lb. per sq. in. was found.* The change of temperature, T, is expressed in degrees Fahrenheit (negative for a drop).

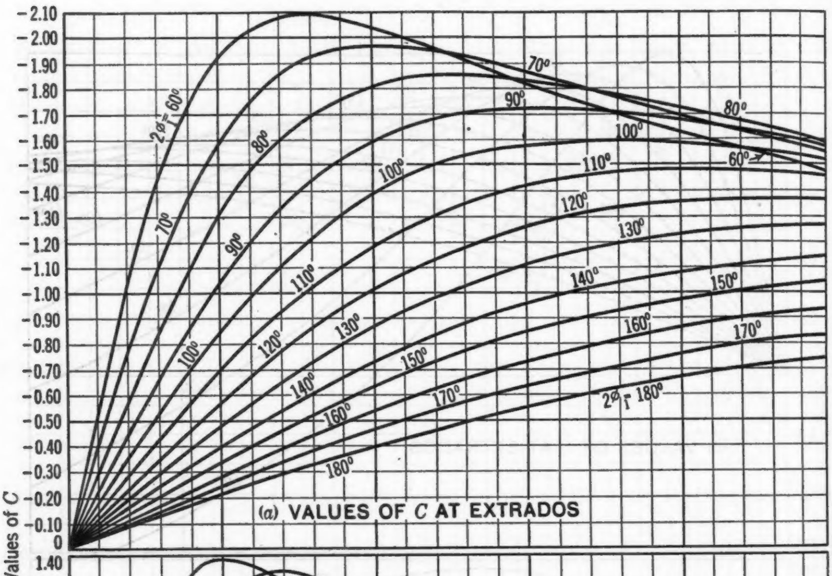


FIG. 2.—VALUES OF COEFFICIENT, C, FOR ABUTMENT SECTIONS.

To illustrate the use of Figs. 1 and 2, assume that $\sigma_c = 350$, $2\,\phi_1 = 110°$, $\frac{t}{r} = 0.10$, and $T = 10°$ (drop). It is required to find the stresses at the

* *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, p. 155.

extrados and intrados, at crown and abutment. The solution is given in Table 1.

TABLE 1.—COMPUTATIONS TO DETERMINE STRESSES WHEN $\sigma_c = 350$, $2\ \phi_1 = 110°$, $\frac{t}{r} = 0.10$, AND $T = 10°$ DROP.

Section. (1)	Cylinder stress, in pounds per square inch. (2)	Coefficient, C. (3)	Rib-shortening stress, in pounds per square inch = Column (2) × Column (3). (4)	Total load stress, in pounds per square inch = Column (2) + Column (4). (5)	Temperature stress, in pounds per square inch, = 15 × 10 × Column (3). (6)	Total stress, including temperature, in pounds per square inch. (7)
Crown, extrados......	350	0.31	108	458	46	504
Crown, intrados.......	350	−0.39	−136	214	− 58	156
Abutment, extrados..	350	−0.70	−245	105	−105	0
Abutment, intrados...	350	0.65	227	577	97	674

The writer is aware that the curves in Figs. 1 and 2 have been preceded by others, devised with essentially the same purpose, and serving it very well. From his point of view, however, the curves herewith present the facts in a somewhat simpler and more directly usable form, particularly in that the coefficient which they give is applicable both to rib-shortening and temperature stress. This reduces the number of charts and functions with which the designer must deal.

The coefficients, C, can be obtained from the curves without difficulty, with a probable error of not more than 1% or 2% of the higher values. The accuracy is, therefore, sufficient both for the purpose of design and analysis of stresses in arches, of given dimensions. They were computed on the basis of the theory given by Professor Cain.* The range of the coefficients, C, in Figs. 1 and 2, is from about -2.1 to $+1.4$ which is very convenient for practical computations.

The coefficients, K, for "thick" arches, were also computed on the basis of Professor Cain's theory.† These values were plotted in Figs. 3 and 4. It should be noted, however, that the values of K correspond to $(1 + C)$ in Figs. 1 and 2. The total load stresses, therefore, are obtained directly by the relation: Total load stress is equal to K times the cylinder stress.

A comparison of the two sets of diagrams shows that for relatively thin arches there in only a small difference between the values of K and the corresponding values of $(1 + C)$. The difference becomes more marked for thicker arches, that is, for the larger values of $\frac{t}{r}$. For designing purposes it is advisable to use Figs. 3 and 4 to find values of K for thick arches. For

* *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 233.

† *Loc. cit.*, Vol. 90 (1927), p. 522.

computing the stresses due to temperature changes Figs. 1 and 2 may be used with close approximation.

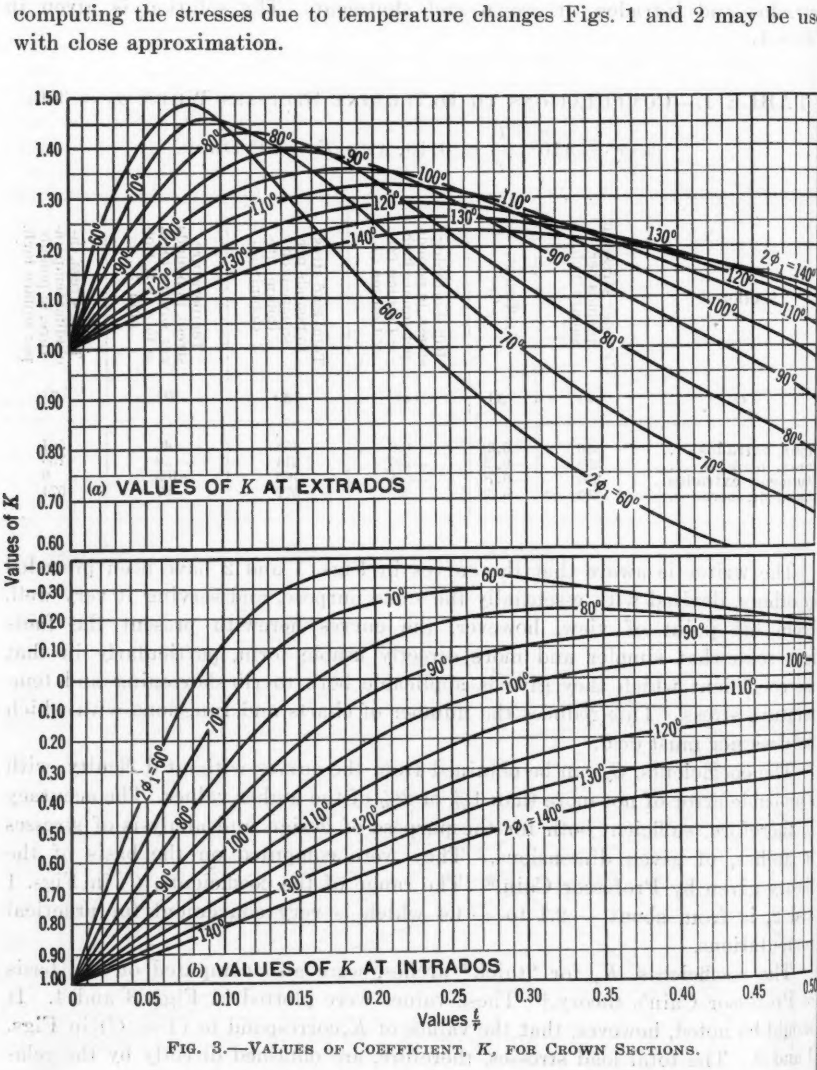


FIG. 3.—VALUES OF COEFFICIENT, K, FOR CROWN SECTIONS.

LAMINATED CONE ARCH DAM WITH FORKED ABUTMENTS

The proposed structure to which reference is taken, is shown in Fig. 5 in a typical layout. It is called Dam No. 1 and is a laminated cone arch dam with forked abutments. This structure has been developed in the pursuit of the ideal, as outlined previously. It is believed that the purposes are accomplished with reasonable success. The diagram shown is a preliminary layout and doubtless would be subject to some modification for a final design. It is given to illustrate general properties only.

For simplicity of layout, and because this is sufficient for preliminary design, the arch sections are made circular in horizontal planes, with radii decreasing from crest to foundation. The crown of the dam is given a pro-

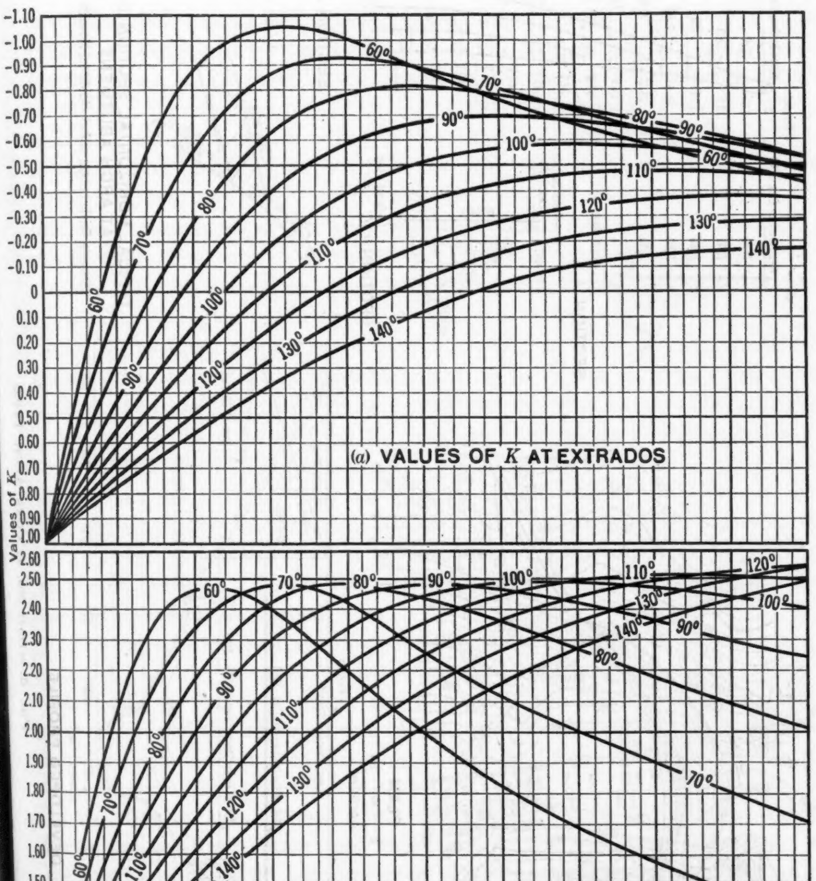


FIG. 4.—VALUES OF COEFFICIENT, K, FOR ABUTMENT SECTIONS.

nounced slope up stream below the crest, making this section overhang down stream from its foundation. The up-stream slope of the arch crown of a cone arch dam involves many of the advantageous features that were used with such marked success in the design and construction of the Coolidge (multiple-

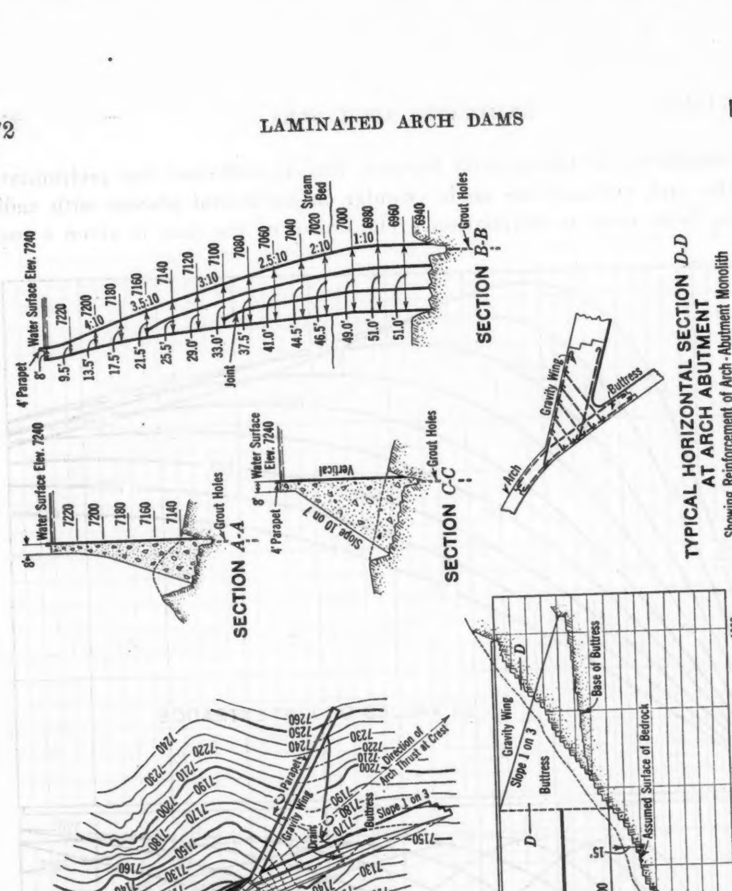


FIG. 5.—DAM NO. 1, CONE ARCH TYPE.

dome) Dam. However, in the cone arch type described in this paper the overhang of the upper arches is very much smaller than in the Coolidge Dam, so that the form work will be much simpler.

It is not necessary, of course, that the shape of the dam conform to any particular geometrical figure, but it is helpful to have in mind a certain fundamental conception, which will be more or less in accord with the actual form. On this basis, in the ideal layout, both extrados and intrados may be conceived as parts of the surfaces of the halves of two elliptical cones, their axes being inclined, and their apexes pointing downward and up stream. The half cone is considered instead of the whole, as otherwise the basic figure would be unsymmetrical, and hence would complicate the geometrical conception, and because the surfaces of the dam will be contained in those of the half cone considered. The axes of these cones will lie in the same vertical plane, and may or may not be either coincident or parallel, depending upon the proportions of the structure and the choice of the designer.

In a practical layout the purpose obviously is to fit the dam most advantageously to the site, not the attainment of perfection of type in geometrical outline; that is, the circular arcs must be adapted to suit the topography. Consequently, departures from the ideal will ordinarily be indicated, and this may be considerable in some cases. Instead of the surfaces of the dam being determined by parts of two cones, the axes of which would be one or two straight lines, this surface may be determined by parts of a number of cones, giving axes which would be a broken line, or lines, or even curved both in plan and elevation. In the latter case the ideal figure may be conceived of as having been bent or warped.

The upper arches of the dam, for such height as may be suitable, are supported by an abutment which is a combination of special buttress and gravity wing-wall, the latter effecting the closure of the reservoir between the ends of the arch and the adjacent side hill at the same elevations. This arrangement is an important feature of this type of dam, and makes possible the attainment of economy without sacrifice of stability.

The use of the buttresses and the sloping crown reduces the radii, and hence the required single arch thickness, to the least practical value. When the height of the dam becomes such that even this thickness given an undesirably high value of $\frac{t}{r}$, the thickness is further reduced by substituting two or more concentric arch rings in contact, for the single arch that would otherwise be required for the direct stresses. In other words, the arch is laminated as required to keep the values of $\frac{t}{r}$ acceptably low, except that the number of laminations of course must be kept within practical limits.

The idea of the cone-shaped arch dam is not new, although it is believed that the present application of the idea is different in several respects from its predecessors. There is nothing to prevent the variable radius arch dam from assuming the form of a cone, and the later developments of this type appear

to be in that direction. However, it is believed that none of these developments of form, to date, has sêcured the maximum obtainable benefits, nor have such developments pointed out the principles through application of which these advantages are to be gained.

The idea of arch laminations, in the manner suggested, is one which occurred to the writer some years ago, but in the sense of actual application in principle, it already has a precedent in a large dam now under construction in France, which will be fully described hereafter.

General Principles of Cone Dam Layout

The actual topography of the site is, of course, the governing feature in any dam layout, which should be such as properly to fit the ground and, at the same time, satisfy the desired theoretical properties. In any actual layout, some compromises must be made between these two aspects of the problem. There are certain general principles, however, which can usually be adopted to control the layout, and which will do so in the case of the typical site to which arch dams are adapted.

Symmetry of the arches about the crown is obviously desirable, and to secure this the axis of the cone should lie in the same vertical plane as the principal axis of the canyon at the dam site; that is, essentially parallel to the foundation contours in the vicinity of the mid-height of the dam. Unless an artificial abutment is provided, the arch element at any elevation ends at its intersections with the foundation contour of the same elevation. Furthermore, in any vertical plane, there should be an easy and suitable transition between the surfaces of elements at different elevations; that is, the position of the axis and the slope of the crown being fixed, the radius at any elevation will be determined by the condition that the arc must pass through the crown point, and must intersect the foundation contours of the same elevation, at a point which satisfies the required outline of the structure. It follows, therefore, that there will be a certain relationship, although indeterminate, between the radii and central angles throughout the dam, the form of each element being determined in a measure by that of the one previously determined. The primary arch element to be established in a dam layout is naturally that one at the crest, and for the reasons previously stated, it is important that this be so chosen as to satisfy the desired conditions in the dam as a whole.

Within practical limits, it is the purpose of this layout to obtain minimum radii of arch elements and reasonably large central angles at the different elevations, but this does not mean absolute minimum radii. It is always possible to decrease the radii by increasing the central angle, but beyond a certain point this would be wasteful. Furthermore, if the radii in the bottom of the dam become too small, the shape of the corresponding arches is not acceptable. Accordingly, the crest arch element should have a fairly large radius, consistent with a central angle of about 80° to 100° between the controlling abutment points.

Functions of Forked Abutments

The primary function of the forked abutments is to decrease the span, and hence the radius that would otherwise be required for the crest and other upper portions of the arch. This should be done without introducing any uncertainties of structural action, or failing to provide in full for the requirements of an arch abutment. Incidentally, the abutments promote symmetry of the arches, which is very desirable, as indicated by the tests on the Stevenson Creek Experimental Arch Dam.

Generally, the two forks of the abutment are placed in such directions relative to the direction of the arch thrust that the thrust is divided into two components supported by the buttress proper and the gravity wing-wall, respectively. The topography at the dam site is usually the guiding factor for the best feasible arrangement of the two forks of the abutment.

For the purposes of design, the buttress is assumed to support its component of the reaction of the abutting horizontal arches. The thickness of the buttress at any elevation is constant and the same as the end of the arch at that elevation. Consequently, the buttress may be considered to be formed by extensions of the horizontal arches.

To insure safety of the buttress against sliding, in addition to the ordinary construction precautions, the layout of the dam should be such that the base of the buttress slopes up the hillside from the end of the arches. From the same point, the top of the buttress may slope downward at the rate of 2:1, 3:1, or any other suitable slope warranted by the conditions.

The height of the buttress, of course, is determined by judgment and economy, but in the writer's opinion there is one consideration which may be given some weight, providing an additional safety factor.

Since the crown of the dam is sloping, the actual load-carrying arch elements will be inclined, and hence only a part of the arch load above the bottom of the buttresses will be transmitted to them. The exact inclination of the true elementary arches cannot be given as a generality, but it will probably be approximately correct to assume them as normal to the slope of the arch crown. If the latter is taken at 4:10, the slope of the normal arches will be 10:4 with the horizontal. The height of the buttress can then be made such that the top inclined arch element (through the crown), if produced, would intersect the base of the buttress and thereby abut directly against the rock foundation. Actually, then, the upper portion of the buttress will receive the reaction only from that part of the dam lying above the uppermost inclined elementary arch. The stability of the buttress, therefore, would appear to be established beyond question, if its down-stream slope is as flat or flatter than, say, 2:1. In fact, the horizontal arch elements abutting against the buttresses will act more or less monolithically with the buttress. The buttress could evidently not be overturned or otherwise overstressed without the entire arch-buttress monolith functioning. Horizontal keyways in the planes of the vertical joints in the arch and between arch and buttresses will tend further to make monolithic action positive. Steel reinforcement extending from the lower parts of the arch upward toward and into the buttresses would

give an additional assurance of monolithic action, but such reinforcement, in general, is not necessary.

That the arches of arch dams function more or less monolithically is clearly proved by the fact that recently in two arch dams one abutment was washed away leaving the upper arches without direct support. Nevertheless, the arches, as such, did not fail.*

The gravity wing-wall, which, together with the buttress proper, forms the forked abutment of the arch, will also effect the closure of the reservoir between arch and side hill. This wall is supporting its share of the direct water pressure by ordinary gravity action and also the component of the arch thrust acting in its direction. The cross-section of the gravity wing adjacent to the arch is usually relatively large and the stresses from gravity action are very small.

The component of the arch thrust acting on the gravity wing-wall, stresses it in an axial direction; that is, perpendicularly to the stresses from gravity action. The gravity wall abuts laterally against the rock foundation, and the arch thrust component in the gravity wall is thereby transmitted directly to the rock. Assuming a properly stepped foundation, a sliding up hill, or an overturning of the gravity wing in a longitudinal direction, due to the arch thrust component, is naturally a rather remote possibility.

Under normal conditions, a relatively small proportion of the arch thrust may be supported by the gravity wall branch of the forked abutment. It is evident, however, that because of its greater cross-section and consequent smaller deformation, the gravity wing-wall would take an increasingly large proportion of the arch thrust if the primary buttress should be subjected to unforeseen overloads such as might cause it, if acting alone, to approach the point of yielding, due either to excessive concrete or foundation stresses.

The gravity wing-wall serves, therefore, with the same amount of material two distinctly different purposes, wherein lies one of the main advantages of the forked type of buttress.

It is true that a so-called gravity tangent may work on the same principle. However, in general, a gravity tangent will not permit of decreasing the span of the arch as much as a forked abutment, except by running it almost parallel to the contour, which is objectionable. Furthermore, a gravity tangent built along a transverse slope is likely to have its base also on a considerable cross-sectional slope. The sliding factor of the gravity abutment would thus be very high. In addition, the arch has a tendency to deflect down stream and, therefore, it exerts an appreciable torsional reaction on the supporting sections of the gravity wing. This action, combined with the shear due to rib-shortening and temperature, results in an additional force upon the gravity wing for which this is not ordinarily designed. In other words, a gravity tangent to an arch dam is probably the weak link, whereas the forked buttresses as here indicated are believed to be the strongest part of the structure.

* *Engineering News-Record,* Vol. 97, October 14, 1926, p. 616.

Effect of Sloping Crown

The arches of the cone type dam are circular in a horizontal plane, but because of the sloping crown of the dam, and in accordance with the law of least work, the actual working elementary arches will be inclined. The effect of this on the probable reaction carried by the buttresses has been noted. It is evident, also, that the inclined elementary arches possess the advantage of carrying their reaction downward into the foundation, instead of horizontally.

The slope of the cone type of dam permits a much greater variation of the arch radii between crest and base than in the ordinary type of variable-radius arch dam. Thus, for a given span and corresponding radius at the crest, the radii of the lower arch elements can be decreased considerably, usually to one-half or less of the length of the radius at the crest. At the same time the central angles of the elementary arches of a cone dam can be kept reasonably large, say, 100° to 110°, or more, resulting in a better distribution of the stresses over the arch section, for reasons previously given.

Another feature of the sloping crown, and consequent overhang of the central part of the dam down stream from the footing, is that this makes the structure particularly suitable as an overflow dam. By confining the overflow to the central part of the crest, the falling sheet of water will be thrown well down stream from the footings, thus reducing the possibility of critical erosion. Depending upon the character of the foundation material and other conditions, it may be permissible to allow the formation of a natural pool; otherwise, a suitable artificial pool may be created by the construction of a down-stream weir.

Effect of Laminations

The planes of lamination are concentric, are essentially smooth and true, and the surfaces are painted with asphalt, or other suitable joint material, permitting minute movements and adjustments between the separate arch sections. The typical manner in which the structure is laminated is shown in Fig. 5.

The structural functioning of a laminated arch dam is approximately as follows: The full water pressure in the reservoir acts on the up-stream face of the up-stream arch lamina. Under such pressure this first arch lamina will be deflected. The immediate contact with the second lamina will put this one under stress, inasmuch as the up-stream arch can not deflect without deflecting and stressing the second arch. This latter will take a proportionate amount of the water load according to its thickness and other dimensions governing its stiffness and strength. In case of additional arch laminæ, a further division of the water load between the first, second, and the additional laminæ will take place. Thus, neglecting for the present the effect of cantilever action, each arch lamina will take such a proportion of the total water load that the deflections of corresponding points along radial lines will be the same for all arch laminæ.

Assume, for instance, a horizontal slice, 1 ft. high, which may be composed of, say, three arch laminæ of equal thickness and span, as shown in Fig. 6. The radial water pressure will be divided between the three arches inversely in proportion to their deflections under a unit of load. The actual deflections, of course, are the same for all three arches, if the compression of the concrete in a radial direction is neglected.

If the radii, thickness, and central angles of the individual arch laminæ were exactly the same, it is evident that the water pressure would be divided equally between the three arches in order to produce the same deflections.

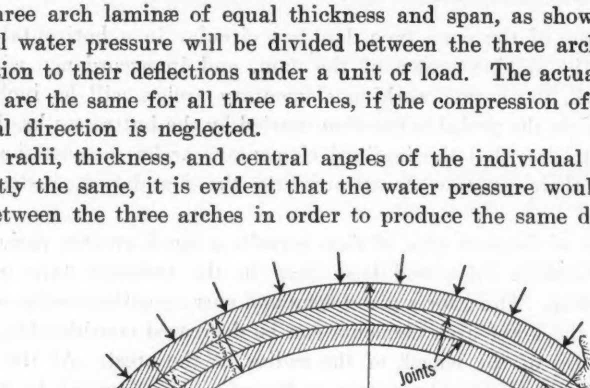


FIG. 6.—HORIZONTAL SECTION THROUGH A LAMINATED ARCH DAM.

However, the arch radii will differ by lengths corresponding to the thickness of the arch laminæ, and the radii of the down-stream arches will always be smaller than those of the up-stream arches. On the other hand, the down-stream arches will usually have a larger central angle than the up-stream arches. Generally, these two factors will tend to compensate each other in the influence upon the deflection of the various arch laminæ and their ability to carry their proportion of the load.

It will be a relatively simple matter to determine in each case the proportion between the thicknesses of the individual arch laminæ, in order to secure approximately the same stresses in all of them.

The joints between the arch laminæ incidentally will subdivide the vertical cantilevers of the dam into cantilever laminæ. The deflection of a cantilever under a given load is inversely proportional to its moment of inertia. In the case of a cantilever, 1 ft. wide, cut by assumed vertical planes from a dam, the deflection due to a given load is inversely proportional to the third power of the thickness. Then, for a cantilever with n laminæ of equal thickness, and assuming further the total load to be equally divided between the n laminae, the deflection of each lamina is increased by $\frac{n^3}{n} = n^2$. In other words, the deflection of the laminated cantilever is increased by the

square of the number of laminations. Consequently, the cantilever elements of laminated arch dams will be more flexible and the bending moments at the base of the cantilevers which in a monolithic dam would be likely to produce cracks between the dam and the foundation at the up-stream face, are greatly reduced. Furthermore, each cantilever element will be bent individually, thus giving a better stress distribution upon the foundation than in a monolithic structure.

The laminating slightly increases the deflection of the arches. The deflection of the vertical elements is increased in proportion to the square of the number of laminations. A greater proportion of the load therefore is thrown upon the arches, especially near the base of a dam; this is desirable because it increases the efficiency of arch action.

The effectiveness of laminating the arches of thick arch dams is evident from the following considerations. Suppose it were practicable to subdivide an arch dam having a radius of 100 ft. into a number of very thin concentric arch laminæ, say, 1 ft. in thickness, each. The dam would then be composed of extremely thin arch rings for which the cylinder formula would be applicable with a relatively small percentage of error. The vertical cantilever elements would be so thin and flexible that they would probably carry only a very small percentage of the load except in the immediate vicinity of the foundation. The cylinder formula, therefore, would be approximately valid for such a structure and the water pressure would be transmitted to the arch abutments by a practically uniform axial compression in the thin arch laminæ. From the theoretical point of view this would result in maximum efficiency, and for a given allowable unit stress would require a minimum of concrete. In practice, this ideal, of course, can not be attained, but it can be approached by providing a reasonable number of laminations.

It is evident that the principle of laminating arch dams may be applied also to arches which increase in thickness toward the abutments as well as to arches of uniform thickness between abutments. In fact, arches which are thicker at the abutments will be most economical, because the critical stress is generally at that point.

There are several ways to provide joints for laminations without unduly increasing the unit prices of the concrete of the arch. For instance, the arch on the down-stream side of the joint may be built one lift of, say, 5 ft. higher than the adjacent up-stream arch; the up-stream forms of the higher arch may then be removed, and the face painted with asphalt or some other suitable material to prevent bond across the joint. Then, the up-stream lamina is poured directly against the face of concrete so treated.

Instead of a coating of asphalt the up-stream face of the down-stream lamina may be provided with a water-proofing membrane, thus giving reasonable assurance that no water will seep through to the down-stream face of the dam. By placing split drain pipes against the membrane at suitable intervals of, say, 20 to 40 ft., any water seeping through the up-stream arch lamina may be drained off readily.

A simple method of producing a cold joint between the arch laminæ, would be to place and securely hold in position, a continuous sheet of impregnated

burlap or elastite and then pour concrete on both sides of the sheet. This will provide a joint, at a minimum interference with concreting.

It is important that the lamination of an arch dam be arranged so that the highest lamina is at the up-stream side of the dam. The other laminæ are placed in steps down stream from the first lamina. The laminæ are not connected with each other, except possibly along the foundation. Any water that might seep from the reservoir through the first arch into the joints between the laminæ, can easily be drained away, if desired. One may also choose to permit a film of water to accumulate in the joints between the different arch laminæ so that each lamina would be subjected to a definite hydrostatic pressure. Any surplus water in the joints would be likely to escape from them over the tops of the different laminæ, or could be drained in a lateral direction at any desired elevation toward the arch abutments. In any case, it is quite obvious that none of the individual arch laminæ could ever be overstressed by excessive hydrostatic pressure in a joint.

Fig. 7 shows the plan, elevation, and cross-sections of a laminated cone arch dam proposed to be built for an irrigation project in California. In order to insure positive and unfailing action of the drainage system in the joint between the arch laminæ a number of overflow openings are provided along the top of the lower arch lamina. Water rising in the joint to the top of the lamina would immediately drain off and thus prevent the lower arch lamina from being subjected to excessive hydrostatic pressure. In addition to these overflow openings drains are provided in the joint leading to an inspection and drainage tunnel extending along the foundation of the arch. From this inspection tunnel the vertical drains in the joint may be opened up, if necessary. Thus, there is no possibility of the down-stream arch lamina ever becoming overloaded through clogging of the drains.

The factor of safety of a laminated arch dam can be ascertained by model tests in a relatively simple manner by loading each arch lamina of the model separately to the point of breaking. The difficulty and expense of securing sufficient static pressure to produce failure of each arch lamina separately, is evidently much smaller than if a model of the complete dam were tested to destruction.

An investigation was made to determine whether the friction in the asphalted joint might prevent the sliding of the arch laminæ upon each other, thus defeating to some extent the purpose of laminating. Assume an elementary horizontal arch with two arch laminæ at a depth of 200 ft. below the crest of a dam. The water pressure at this depth is $62.5 \times 200 = 12\,500$ lb. per sq. ft. The up-stream lamina is assumed to carry one-half this pressure, the other half (6 250 lb. per sq. ft.) being transmitted across the joint to the down-stream lamina. Assuming a coefficient of friction in the asphalted joint of 0.5, the frictional resistance in the joint, therefore, would be $0.5 \times 6\,250 = 3\,125$ lb. per sq. ft. (22 lb. per sq. in.). This is rather small as compared to the difference in stress in the concrete on opposite sides of the joint, which may be as much as 200 to 300 lb. per sq. in. It is evident, therefore, that the friction in the asphalted joint between the arch laminæ

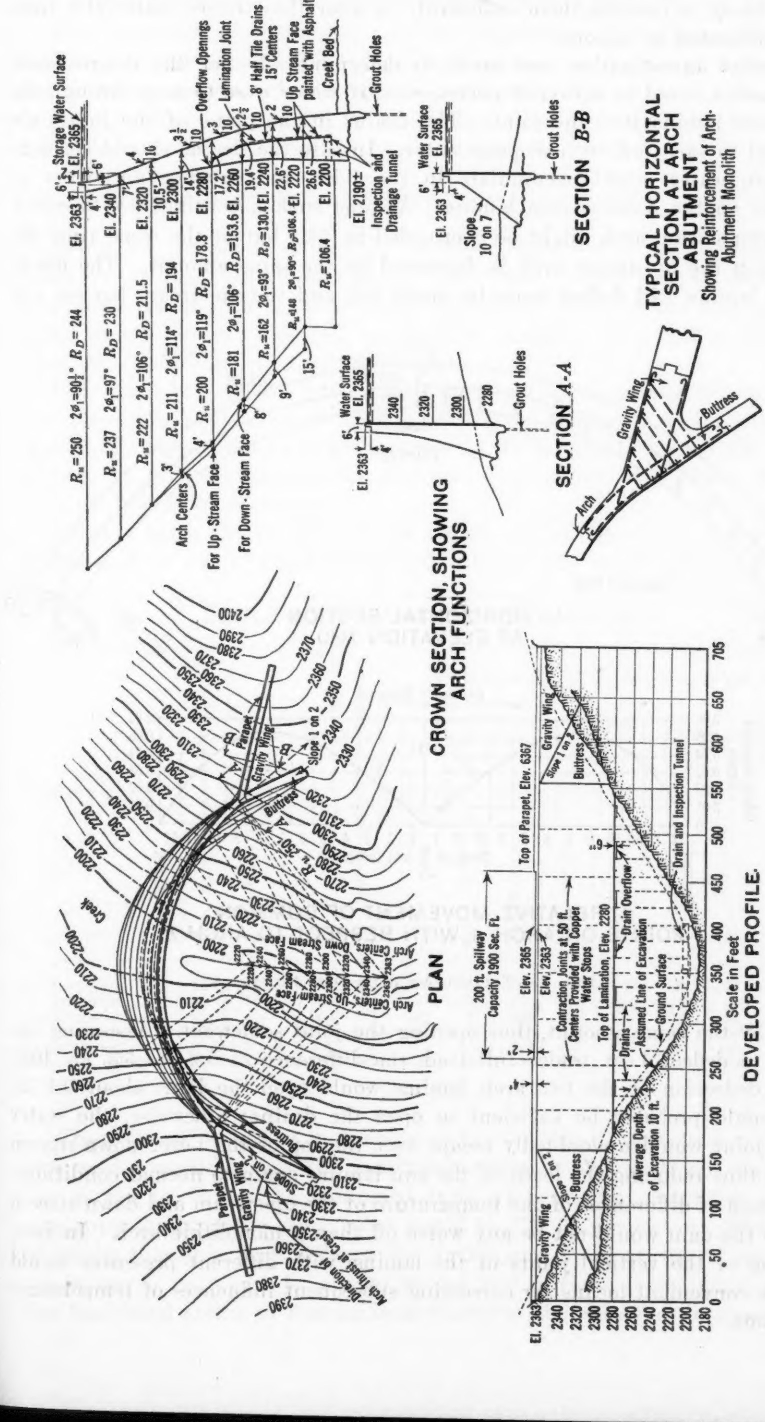


FIG. 7.—PLAN AND SECTIONS OF DAM NO. 2.

is not likely to restrain them sufficiently to alter the stresses materially from those indicated by theory.

Another investigation was made to determine whether the down-stream arch lamina could be seriously overstressed if water were to seep through the up-stream lamina into the joint. The drains in the plane of the joints are provided to carry off such seepage water. In case the drains should cease to function, water might accumulate in the joint and produce an excess of pressure on the down-stream lamina. Assume such a contingency to occur. The down-stream arch might be overloaded by 5%, but at the same time the load upon the up-stream arch is decreased by the same amount. The down-stream lamina will deflect more by about 5% and the up-stream lamina will

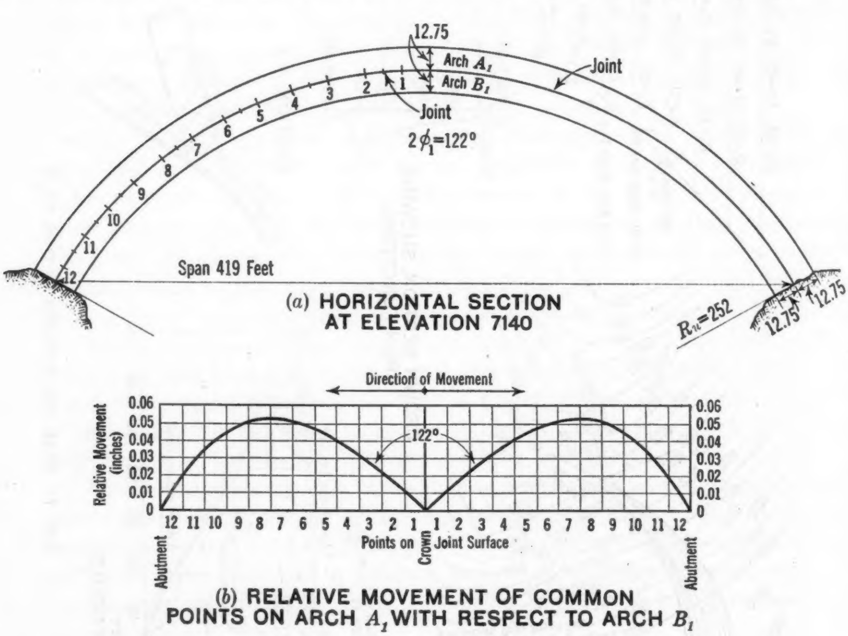


FIG. 8.—TYPICAL ARCH OF DAM NO. 1.

recede by the same amount, thus opening the joint somewhat. Assuming the arches to deflect 1 in. under full load, the difference of 5% + 5% = 10% of the deflection of the two arch laminæ would open the joint about 0.1 in. This would probably be sufficient to open the drains; otherwise, the water in the joint would undoubtedly escape over the top of the lower down-stream lamina thus reducing the loads of the two laminæ again to normal conditions.

In case of differences of the temperature of the up-stream and down-stream laminæ the dam would not be any worse off than a monolithic arch. In fact, grouting of the vertical joints of the laminæ with different pressures would afford a convenient means for correcting subsequent influences of temperature variations.

Determination of Maximum Arch Thickness

The maximum allowable arch thickness, and hence the number of laminations required at any elevation, depends, of course, on the design assumptions and working stresses. Assuming the preliminary design to be based on a uniform stress of 350 lb. per sq. in. by the cylinder formula, and assuming the actual critical stress to be the tension at the abutment, it is indicated that the maximum uniform thickness is reached when $\frac{t}{r}$ is about 0.15, assuming $2\phi_1$ at about 90 to 100 degrees. Recourse to laminating is not necessary at this point, however, as the critical stress can be reduced by thickening the arch at the abutment. The point at which the first or subsequent laminations are required, is, therefore, a matter of judgment dependent upon the conditions, to a certain extent.

Fig. 8 (*a*) is a typical horizontal section through the dam shown in Fig. 5 at Elevation 7140. The arch at this elevation is thicker at the abutments than at the crown, but in order to simplify the computations for the present purpose, it will be assumed that the arch laminæ are of uniform thickness from abutment to abutment.

Table 2 gives the stresses at the crown and abutments in the two arch laminæ and, by way of comparison, also in the unlaminated arch.

TABLE 2.—Comparison of Arch Stresses.

Arch. (See Fig. 8.)	Arch thickness, t, in feet.	Radius at center line, r, in feet.	$\frac{t}{r}$.	Central angle, $2\phi_1$, in degrees.	Arch Stresses, in Pounds per Square Inch. Cylinder.	Rib-Shortening. Crown.	Rib-Shortening. Abutment.	Cylinder Plus Rib-shortening. Crown.	Cylinder Plus Rib-shortening. Abutment.	10° Drop of Temperature. Crown.	10° Drop of Temperature. Abutment.	Total for Load and Temperature. Crown.	Total for Load and Temperature. Abutment.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
A_1, Extrados.......	12.75	245.6	0.052	122	429	64	−129	493	300	25	−50	518	250
A_1, Intrados	12.75	245.6	0.052	122	429	− 70	125	359	554	−27	48	332	602
A_2, Extrados.......	12.75	232.9	0.055	122	429	67	−135	496	294	26	−52	522	242
A_2, Intrados	12.75	232.9	0.055	122	429	− 75	131	354	560	−29	50	325	610
Monolith, extrados.	25.50	239.8	0.106	122	429	114	−249	543	180	44	−96	587	84
Monolith, intrados.	25.50	239.3	0.106	122	429	−140	232	289	661	−54	89	235	750

From the values given in Column (14) of Table 2, it is seen that in the arch under consideration the maximum stress at the abutment intrados is 610 lb. per sq. in. in the laminated arch, while in the monolithic arch the maximum stress at the same point would be 750 lb. per sq. in. The process of laminating this arch reduces the maximum stress at the abutments by 140 lb. per sq. in., which is between 20 and 25% in favor of the laminated arch. For laminated arches of non-uniform thickness the reduction in stress, or the

economy of material in case of the same size of maximum stress that may be obtained by laminating, is generally about in the same proportion as for arches of uniform thickness, such as those considered in the preceding example.

Fig. 8(*f*) gives the movement of Arch A_1, relative to the arch, A_2, in the plane of the joint between the two arches, due to a change of water pressure from no load to full load. For this purpose the stresses and corresponding strains at the intrados of Arch A, and at the corresponding points across the joint, at the extrados of Arch A_2 were calculated and the elastic deformations at the arch surfaces added from abutment to abutment. A modulus of elasticity of 3 000 000 lb. per sq. in. was used in the computations.

Each arch was divided into twenty-four sections of equal length, and the stresses were computed on radial sections through the center of these lengths, as indicated by the numbered points in the diagram. The stresses thus found were assumed to be the average for each increment, ΔS, of length, and the deformation computed accordingly, on the intrados of Arch A, and the extrados of Arch B. Beginning at the abutment, these deformations for each arch were progressively added. The difference in these sums, for any point, gives the movement of this point on the intrados of Arch A with respect to the same point on the extrados of Arch B, which is shown graphically in the diagram.

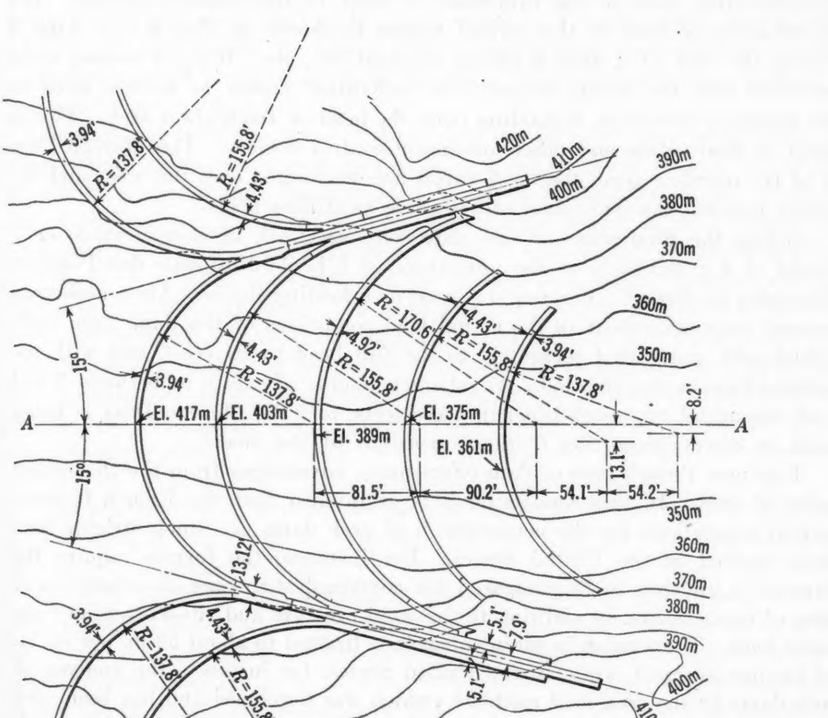
Thus, for a laminated arch of the shape and dimensions shown in Fig. 8(*a*) subjected to stresses, as given in Table 2, the maximum relative movement of the two arches in the plane of the joint would be about 0.05 in. This is very little and may take place in an asphalted joint without difficulty. The maximum sliding movement in the joint of 0.05 in. will occur only in case of a change from no load to full load conditions, or *vice versa,* and accompanied by a change of temperature corresponding to that assumed in Table 2. For most dams, such conditions obtain not oftener than once or twice each year.

French Dam Utilizing Principle of Lamination

In connection with the writer's proposed lamination of thick arch dams, an account of a high dam now under construction in France, in which the same principle has been utilized, should be of interest. Since this structure represents such a bold departure from precedent it is believed that a fairly complete description will be in order.

The dam is at Marège, on the Dordogne River, in France.* The lamination is effected in a rather different way from that proposed by the writer, but the purpose in view, and except for slight differences, the general structural effects are essentially the same. The dam, as shown in Fig. 9, will be be composed of five thin arches of reinforced concrete, the thickest being only 4.9 ft., and separated by distances varying from about 59.5 to 73.8 ft. The tallest arch will be about 230 ft. high, the others being lower in steps of about 46 ft. each. Assuming reservoir full, the basins between the individual arches are filled with water, so that each arch has to carry a maximum load

* *Revue Générale de l'Electricité,* October 29, 1927.



(a) PLAN

(b) SECTION A-A

(c) DIAGRAM OF LOAD DISTRIBUTION

FIG. 9.—DORDOGNE RIVER DAM AT MARÈGE, FRANCE, PLAN AND SECTIONS.

corresponding only to the difference of head in the adjacent basins. The distribution of load to the several arches is shown in Fig. 9 (*c*), Arch 1 taking the load, W_1; Arch 2 taking the load, W_2, etc. It is, of course, quite essential that the basins between the individual arches be always filled to the necessary elevation, depending upon the head on the highest arch. This is done by float-valves and other automatic control devices. The Marège Dam is of the overflow type; the flood-waters are discharged over the crest and the basins between the individual arches serve as stilling-pools.

Before the final plans of this dam were accepted, tests were made on a model of 1 : 100 scale at the laboratory of L'Ecole Nationale des Ponts et Chaussées in Paris.* Mercury was used as a loading liquid. After numerous general tests equivalent to the actual load conditions of the dam, each individual arch was tested separately to the full head to its crest, and with the intermediate basins emptied. An ultimate factor of safety of between 3 and 4 of the model was established in these tests, and the Marège Dam is being built in direct proportion to the dimensions of the model.

This new French type of dam offers many advantages from the theoretical point of view. In this connection it is of interest that the French Governmental regulations for the construction of arch dams are much stricter than those applied in the United States. For instance, the former require the stresses in an arch to be computed for combined shrinkage of concrete and drop of temperature, in addition to the axial pressure and rib-shortening from water load. The tension in plain concrete is limited to about 52 lb. per sq. in. (4 kg. per sq. cm.), while in the United States, for instance, the analysis of arch dams by the trial-load method† (which was developed and has been used extensively by the U. S. Bureau of Reclamation), not only contemplates high tensile stresses, but numerous consequent cracks in the arches and cantilever elements of an arch dam.

Except in special cases, under the French regulations it would be practically impossible to design and build a high and thick arch dam of the ordinary constant-radius or constant-angle arch type. An analysis would almost certainly indicate high tensile stresses, prohibited under the French rules. Therefore, the French engineers decided that the logical step would be a change in type of structure, to one susceptible of rigid analysis. The new type of arch dam is the consequence.

Several years ago a Swiss engineer, R. Maillard,‡ proposed a similarly stepped-off type of construction, not only for arch dams, but also for structures of the multiple-arch and Ambursen type.

The Swiss and French designs probably represent the ultimate attainment in the application of pure logic to the problem of high arch dams, resulting in a design of maximum structural simplicity and determinateness. The striking feature of their plan is that not only is the total load distributed

* A description of these model tests and a photograph of the model are given in *La Revue Industrielle*, July, 1927, p. 340; see, also, *Engineering News-Record*, August 30, 1928, p. 311, and *Western Construction News*, July 25, 1928, p. 465.

† "Analysis of Arch Dams by the Trial-Load Method," by C. H. Howell, M. Am. Soc. C. E., and the late A. C. Jaquith, Esq., *Transactions*, Am. Soc. C. E., Vol. 93 (1929), p. 1191.

‡ *Schweizerische Bauzeitung*, April 14, 1928, p. 183.

to a series of independent arches, but it is so subdivided that, except the lowest arch, each one is uniformly loaded vertically for the greater part of its height. This, obviously, tends to a great simplification of structural action, particularly when buttresses are used, keeping the length of many arch elements constant, as they have done.

Another outstanding characteristic of this design is the very small foundations which it requires, except in case of a great depth of loose overburden. It will be observed that the aggregate width of foundations of the Marège Dam is only about 30 ft., and this for a dam the effective height of which is very nearly 230 ft. This is, of course, a direct consequence of the very thin arch sections, and the assurance that the vertical foundation reactions will be fairly uniformly distributed. Judging from the topography, in connection with the sectional elevation, the depth of excavation is also very moderate. This, of course, might be accounted for by excellent foundation conditions, but it might also be due to the fact that the maximum argument for leakage through the foundation is only the difference in head on the two sides. This same consideration is an important justification for the relatively extreme thinness of the arches.

The adaptability of this particular type of layout to dam sites in general, may be open to some question, and the plan would probably require modification to suit varying conditions.

In spite of the great apparent theoretical advantages of the design, it is such a radical departure from present practice that engineers elsewhere may be slow to adopt it. Also, contrary to its actual functions, the structure as a whole presents a rather complex appearance, which may be regarded with disfavor; the extremely thin concrete sections may be looked upon with suspicion, and the form work required may be an adverse feature.

The writer's proposed method of lamination, while falling short of the ideal theoretical accomplishments of the French design, perhaps sufficiently achieves the desired results, more harmoniously with American practice and conditions. Particularly from the standpoint of appearance, the finished structure will seem to be a massive monolith, not unsimilar in outline to structures representative of advanced American practice.

Reduction of Cantilever Influence in Design

The several design features proposed by the writer will make it possible to obtain relatively thin arches throughout the dam. As pointed out, the cantilever elements will become comparatively ineffectual. Consequently, the elementary arches will not only take by far the greater part of the load, but this will be distributed fairly well in direct proportion to the water pressures; that is, horizontal elementary arches would carry approximately a uniform radial load. The reason for dividing the total load between arches and the cantilevers, in the case of thick arches, is not merely because this produces refinements of design from the standpoint of economy, but because the cantilever action may result in more unfavorable conditions than if they were absent. In other words, if the cantilever action were neglected, not only would the resulting analysis be entirely erroneous, but it might be actually unsafe.

Evidently, the thicker the arch the greater the necessity for considering cantilever action; conversely, the thinner the arch, the greater the assurance that this may be omitted without jeopardizing the safety of the analysis, which will at least approach the actual conditions. While the writer does not favor discarding any proper refinements of analysis, it is believed that in the proposed design, the elementary arches might be assumed to carry the entire water pressure, and yet give essentially the same dimensions of structure as if the cantilevers were considered. At least, the preliminary design would require the minimum of subsequent modification. The design, of course, may be checked and altered if necessary, by the trial-load method,* investigating a suitable number and combination of elementary cantilever and arch elements.

To avoid an undesirable reversal of stress at the base of the dam, and to make the action of the vertical elements more determinate, a temporary hinge may be provided during construction, and later grouted, as recommended† by Victor H. Cochrane, M. Am. Soc. C. E. This, of course, would tend to diminish the cantilever action even more.

The Railroad Canyon Cone Arch Dam

As an example of the actual application of the design principles proposed by the writer (except that of lamination), there follows a description of the Railroad Canyon (cone arch) Dam, built in 1927-28, on the San Jacinto River, near Elsinore, Calif. (See Figs. 10 and 11.) The height and layout of the dam were such that laminations were not required. The properties of this design, except laminations, are generally the same as those that have been described and discussed in detail in connection with the typical design, Fig. 5.

The dam is of the overflow type, the spillway over the central portion of the crest being designed for 30 000 sec-ft. The maximum recorded flood at the site is 15 000 sec-ft. The crest overflow will be spilled into a stilling-pool formed by a small arch, 20 ft. high, built about 50 ft. below the dam. (See Fig. 10.) Except immediately adjacent to the crest and the base, the crown slope is 10:4, which gave a layout suitable to the topography.

Those portions of the arches adjoining the abutments, and also both forks of the abutments were strongly tied together with 1½-in. round reinforcing bars. The buttress was designed to take the computed arch thrust, but, as previously stated, it seems evident that failure could occur only by the simultaneous crushing or sliding of both the buttresses and the wing-wall. This affords a very large safety factor, over and above that provided in the design.

The highest axial stresses in the horizontal arches due to the water pressure are about 360 lb. per sq. in. The arches are of uniform thickness at the upper elevations, but are thickened toward the abutments, in the lower portion of the dam. In the design, the maximum allowable compression was taken at 600 lb. per sq. in., and the maximum tension at 100 lb. per sq. in., including the stresses due to rib-shortening, which are the limits set by the State Engineer of California.

* *Transactions*, Am. Soc. C. E., Vol. 93 (1929), p. 1191.

† *Loc. cit.*, Vol. LXXXVII (1924), p. 378.

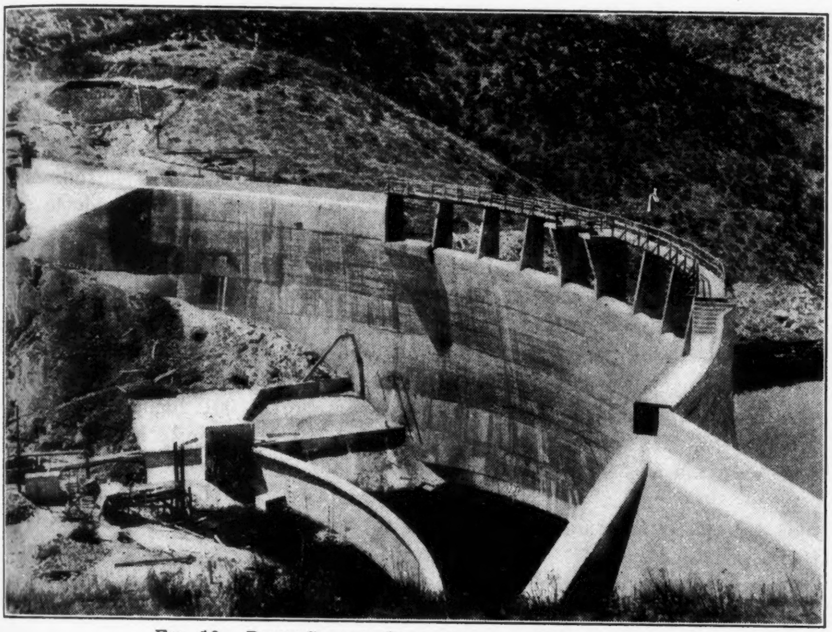


FIG. 10.—DOWN-STREAM FACE, RAILROAD CANYON DAM.

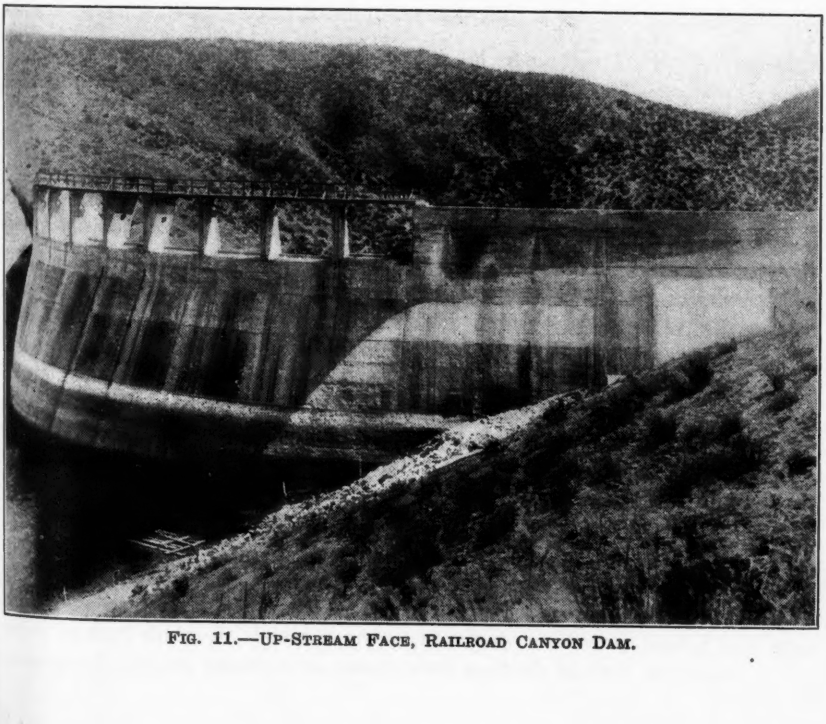


FIG. 11.—UP-STREAM FACE, RAILROAD CANYON DAM.

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The bed-rock at the dam site is slate. In the bottom of the canyon and for a short distance upward the rock is extremely hard, in places almost as hard as flint. At the upper elevations it is softer and fractured into small pieces. Where the shattered rock was encountered the excavation was varied to a maximum of 35 ft. below the original surface of the ground, and the concrete abutments of the arch were widened to give an average pressure on the rock of about 10 tons per sq. ft.

A fault line in, and approximately parallel to, the river bed, crosses the dam site, but this is considered "dead" in an opinion given by Roy G. Mead, Consulting Geologist, who made a report on the geologic conditions at the dam site. Special care was taken in the excavation at the fault line by grouting and deep cut-off work. In addition, numerous grout holes were drilled into the rock below the base of the dam, and filled with cement grout under pressures of about 100 lb. per sq. in. A cut-off wall was built along the up-stream face of the dam for further safety against seepage. The gravity wing-walls were designed for the hydrostatic head assumed to act over 50% of the area of the base.

Before deciding upon the dam for this location, comparative designs and estimates of cost were made for five types of dams. These studies indicated that the laminated cone arch type would save approximately 20% in cost over the next cheapest. It contained less than 50% of the concrete necessary for a straight gravity dam.

Acknowledgments

The writer is indebted to S. M. Cotten, M. Am. Soc. C. E., for valuable aid in the preparation of this paper.

Conclusions

The type of arch dam described in this paper involves three features which promote both simplicity of design and economy, namely, forked abutments, sloping crown of the arch, and laminations:

(1) The so-called "forked abutments" take the thrust of the upper portions of the arch. Each abutment consists of a buttress and a gravity wing-wall poured integrally, thereby forming an arch abutment of large cross-section and great resistance to shearing, overturning, and sliding. The forked abutments reduced the span of the arch and permit the use of relatively short arch radii and a correspondingly small thickness of the arch.

(2) A pronounced sloping of the crown of the arch, secures short radii and large central angles for the arch elements below the crest. The small overhang of the arch in the center will keep the upper arch elements under light compression when the reservoir is empty, thus preventing the opening of the construction joints.

(3) The sub-division of the total arch thickness into two or more arch laminæ, keeps the arches relatively "thin" and reduces greatly the cantilever action so that the arch elements are more nearly uniformly loaded and may be designed without recourse to complicated methods of arch analysis.

These three features may be used to advantage either singly, in combinations of two, or all three of them, in the design and construction of arch dams at most sites. The resulting structures will involve arches of high efficiency, subject to simple, but nevertheless comparatively rigid analysis. The forked abutments are structural elements with a high degree of design and reserve strength. They can be arranged at many sites in such a way that they improve greatly the conditions as to span, symmetry, and economy of the main portion of the structure, namely, the arch.

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THE CHESAPEAKE AND DELAWARE CANAL

By Earl I. Brown,* M. Am. Soc. C. E.

Synopsis

This paper is a record of the construction stages of the Chesapeake and Delaware Canal. The history of the project from 1764 is outlined, with a description of successive developments to 1927. The paper contains quantities, yardage, prices, total costs, and methods of procedure in connection with dredging operations. It also includes descriptions of several of the bridges that now cross the canal. The paper closes with a brief outline of hydraulic theory as applied to the project.

Introduction

The Chesapeake and Delaware Canal was constructed during the period, 1825-1829, by the Chesapeake and Delaware Canal Company, organized for that purpose under authority of special legislation enacted by the States of Maryland, Delaware, and Pennsylvania. Each of these States, as well as the Federal Government, subscribed for stock in the Company.

The location selected (after prolonged investigations, and abortive attempts at construction on other locations) was an east and west line connecting St. Georges Creek on the Delaware River about 40 miles below Philadelphia, Pa., with Back Creek on Chesapeake Bay, about 54 miles east of Baltimore, Md. The length of the canal was about 14 miles, and the summit level 17 ft. above mean low water in the Delaware River. As originally constructed, the locks were 22 ft. wide and 96 ft. long, with a depth of 8 ft. over miter-sills. Extensive improvements were made about 1850 to 1855, when new locks were constructed, 24 ft. wide and 220 ft. long, with a depth of 10 ft. over miter-sills. The original Company operated it as a toll canal for 90 years.

On August 13, 1919, the canal was taken over by the United States for the purpose of replacing it with a new sea-level canal of somewhat larger dimen-

Note.—Written discussion on this paper will be closed in **August, 1930**, *Proceedings*.

* Col., Corps of Engrs., U. S. A.; Engr., 8th Corps Area, No. 1 Staff Post, Fort Sam Houston, Tex.

sions. The enlargement was effected during the years, 1922-1927, and normal traffic was maintained practically uninterrupted during that period.

Reconstruction involved the excavation of about 16 000 000 cu. yd. of earth, the construction of four highway bridges and one railway bridge, the relocation of the Delaware end of the canal, the construction of tide dikes, jetties, and other works of a miscellaneous character.

The deepest cut was through the summit divide where the general land elevation is 80 to 85 ft. above sea level. Practically all excavation was successfully done by pipe-line dredges. Most of the spoil was deposited in specially prepared basins on the adjacent banks, involving lifts of 80 to 95 ft.

All new bridges are of the vertical lift type, electrically operated. They were designed for heavy military loads, and the foundations were built in contemplation of a future deepening of the canal to as much as 30 ft. The navigable span provides an opening of 175 ft. clear width, and a vertical clearance of 140 ft. above mean low water, when open.

The canal was opened to traffic at sea level on May 14, 1927, with approximately full project depth of 12 ft. at mean low water and a bottom width of 90 ft.

Historical

The two great bays, the Delaware and the Chesapeake, make deep indentations into the Eastern Coast of the United States. The northern end of Chesapeake Bay, inclining to the eastward, approaches close to Delaware Bay at a point where the latter, as the Delaware River, attains its farthest westerly meander; the distance between them at this point is only 20 miles. Both sides of the intervening peninsula are again deeply indented by tributaries which often approach to within a few thousand yards of each other at their head-waters. Chesapeake Bay, in this upper region, is marked by shores that are bluff, steep, and quite free from marshes; the outlets to its rivers are open and clean; the water is broad and fairly deep; the bottom is hard and usually sandy.

Delaware Bay has quite different characteristics, broad marshes border its shores with a width of 3 to 4 miles at places. The banks are so low that 2 to 3 ft. of water cover them at high tides; the outlets of its tributaries are obstructed by mud-bars. The high ground between the two bays is rather low in the broader parts of the peninsula farther to the southward, but at the canal it is 80 to 90 ft. above sea level.

The short distance at the upper end of the peninsula early invited plans for connecting the two bays by a navigable channel so that commerce would not be compelled to follow the sea route around, or make the portage across.

The second survey made for a canal in America is said to have been for connecting these bays near the present location. This was about 1764. Another survey followed in 1769, but the project was interrupted by the Revolutionary War. It was begun again in 1799, after the passage of an act by the Legislature of Maryland incorporating the Chesapeake and Delaware Canal Company. Similar acts were passed by Delaware and Pennsylvania and each State contributed toward the purchase of stock. The Federal Gov-

ernment likewise took an interest in the matter and subscribed for stock to the amount of \$450 000. Pennsylvania contributed \$100 000; Maryland, \$50 000; and Delaware, \$25 000. The cost estimated at that time was \$2 250 000, and the remainder (\$1 625 000) was to be obtained by popular subscription.

The first route selected (see Fig. 1) was an unfortunate one, and the plans (which provided for a canal with fourteen locks from Elk River to Christiana River) had to be abandoned for financial reasons as well as for lack of a suitable water supply. The City of Philadelphia opposed it because of an idea that it was unduly favorable to Wilmington, Del.

Actual construction operations had been begun in 1804, but were suspended after about two years. In the meantime additional studies were made, particularly from 1822 to 1824. These led to the definite abandonment of the old route and the selection of the one actually followed, which is substantially that still in use. The Company was re-organized and refinanced, so that it was able to begin active field work in 1824 and to complete the work in 1829. It was a large undertaking for the period.

Stung by the rapid growth of Philadelphia and New York, N. Y., the City of Baltimore began to take an interest in finding a shorter route to the sea and to northern ports. In 1871 a commercial convention was held in Baltimore and a movement toward establishing a ship canal between Chesapeake and Delaware Bays was inaugurated. It appeared that a saving of approximately 320 miles would be effected by such a canal. As a result the House of Representatives requested information of the Secretary of War. In 1882, the River and Harbor Act directed that a survey be made of various routes. Several District Officers and Boards reported on possible alternate routes, but all finally came to the same conclusion, that the existing canal route was the most feasible.

As a net result the United States finally decided to buy the canal and to re-construct it so as to eliminate locks and increase its capacity. The immediate reason was to use the canal as a link in a proposed intra-coastal system along the Atlantic Coast. However, it has been generally agreed that the time would probably soon come when a ship canal along this route will be a necessity. This thought has guided all the working plans for the reconstructed canal.

Description

The canal was acquired by the United States on August 13, 1919, for the purpose of constructing along its route a sea-level barge canal, with a depth of 12 ft. at mean low water for a bottom width of 90 ft.

The original canal had its entrance at Delaware City, Del., about 40 miles south of Philadelphia (see Fig. 1). Here a tide-lock maintained a canal level of 7.6 ft. above mean low water in the Delaware River. Thence, the canal extended between banks, partly natural and partly artificial, for a distance of 4.3 miles to the Town of St. Georges, Del., its level at some of the intermediate points being above that of adjacent marshes. Because the banks become higher near St. Georges the canal level there was raised 10 ft. by

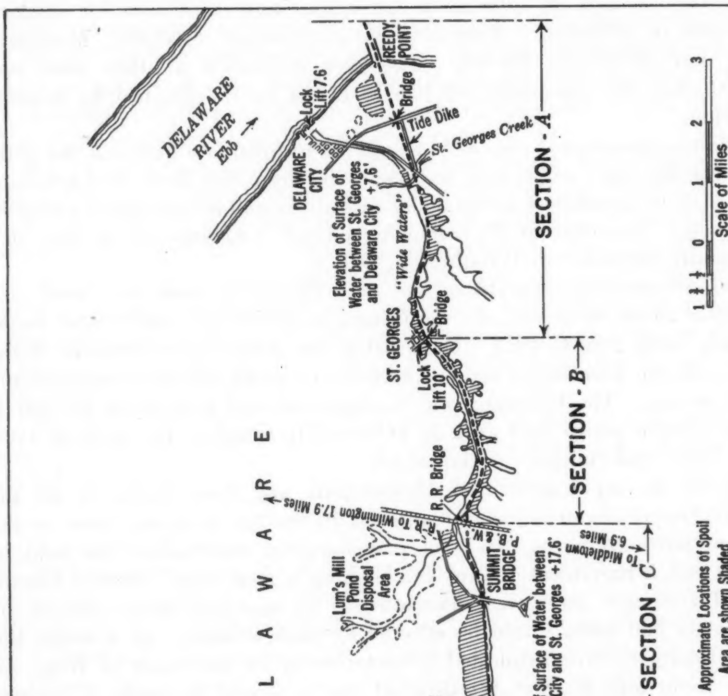

FIG. 1.—CHESAPEAKE AND DELAWARE CANAL: LOCALITY SKETCH.

another lock. Thence, it continued for 9.3 miles to Chesapeake City, Md., where a descent of 15 ft. was made by a third lock to the level of Back Creek, an arm of Elk River and one of the upper branches of Chesapeake Bay. The old canal had a depth of 10 ft. and a bottom width of 36 ft.; the locks were 220 ft. long in the chambers, and 24 ft. wide. This limitation in width was even for many years a more serious handicap than lack of depth.

No natural streams of any consequence flow into the summit level. To supply water for lockages, leakage, evaporation, and seepage, a pumping plant was located at Chesapeake City, to raise water from Back Creek up into the summit level, from which it was drawn as needed. Originally, this comprised a single water-wheel, about 35 ft. in diameter, with spiral buckets. The water was taken up by the scoop-like lip of the buckets on the outer circumference of the wheel, and, as it was lifted, flowed inward toward the axis, and discharged laterally. The capacity of this plant, operating continuously, was about 40 000 000 gal. per day. This was later increased to 100 000 000 gal. per day by installing a 24-in. and a 36-in. centrifugal pump.

At the summit level the average ground surface is 85 to 90 ft. above the sea. This required a cut of some magnitude, about 4 miles long. Elsewhere, the cuts were moderate.

The first Federal appropriation for reconstructing the canal was $3 000 000, of which $2 514 000 was paid the owners as an upset price for all their property, rights, and appurtenances, leaving only $480 000 immediately available for active operations. Later, allotments totaling approximately $8 000 000 were made as needed to complete the work.

The original (1916) estimated cost of reconstructing the canal to project dimensions was $8 000 000. In 1919, the cost of labor and material increased so considerably that a new estimate of $13 000 000 was made. The actual cost of the work up to June 30, 1927, was $10 060 000.

Character of Work

The work to be undertaken resolved itself into four distinct classes: Excavation, bridges, jetties, and miscellaneous, including such work as surveys, disposal basins, drainage, dikes in low marsh lands, changes in existing roads, removal of old locks, etc.

The first two classes were done by contract; the execution of the last two involved many uncertainties as to conditions, quantities, and other requirements inconsistent with definite specifications, and this led to the use of force account.

In planning all operations certain primary considerations had to be kept in mind: First, in authorizing the work, Congress imposed the condition that traffic should be hindered as little as possible during reconstruction (depending on the season, thirty to fifty vessels pass through the canal per day); second, the spoil from excavation should be disposed of near-by; third, in the deep cut section, experience showed that landslides might be expected to recur on a large scale; and, fourth, all land traffic with the Delmarva (Delaware-Maryland-Virginia) Peninsula crossed the canal on the various bridges, and these communications could not be broken.

Planning the Work

In planning the excavating operations, it was necessary to decide between wet excavation, or steam-shovel work. This involved an extended study of the relative costs. The canal was divided into four approximately equal sections in each of which the work was of about the same character as to probable unit cost of excavation.

A new eastern entrance to the canal was to be provided, because the old passage at Delaware City was defective in that it debouched into a secondary channel, which, while deep enough for the present project of 12 ft., would be insufficient and would involve a greatly increased volume of work should the waterway ever be enlarged to a ship canal. A second reason is that the new entrance is placed south of Fort du Pont, one of the defenses of the Delaware River, and this adds to the military strength of that point. The new line joins the original canal about two miles inland.

In the light of all available facts dredging (without first lowering the water in the canal), appeared to be the most satisfactory method of excavation. This would minimize the occurrence of slides. By cutting the bottom to its new proposed elevation (— 12 ft.) and holding the summit level at + 17.6 ft., a depth of water of almost 30 ft. was provided. Slides might then flow with little danger of obstructing the existing traffic. The use of dredges was objectionable because of the difficulty of finding suitable near-by dumps for spoil, and of getting a sufficiently powerful dredging machine through the locks and into the canal.

To dispose of the material conveniently, it was decided to purchase about 300 additional acres of land adjacent to the canal, and to build large earthen dikes around this tract to form spoil basins. The high lift for the pumps (ranging from 60 to 90 ft. above the canal level) precluded any but powerful machines, and it was believed that even then "boosters" would be required with each machine.

A method of introducing large dredges into the canal was devised at Chesapeake City. Two coffer-dams of steel sheet-piling were provided in two sides of the saving basin* adjacent to the canal. By removing one of these coffer-dams, communication was established at tide level with Back Creek. The dredge was placed in the basin, the coffer-dam was restored, and the water in the basin was raised to canal level. Then, access to the canal was gained by removing the other coffer-dam. Altogether, six dredges were taken into and out of the canal in this manner. In making arrangements for excavation contracts, small sections, including the existing and proposed sites for bridges, were excluded.

One railway, one secondary highway, and three main highway, bridges were to be provided. Although the sea-level project contemplated only a barge canal of limited depth and width, it was conceded that such limited dimensions must be regarded as a relatively temporary stage in the canal's growth and that future enlargement would be required to allow the passage of large sea-

* A saving basin is a basin provided alongside a lock into which the upper half of the prism of water between upper and lower levels may be drawn in emptying the full lock, and there preserved for use in filling the lower half when the lock is next filled. The net loss of water in a lockage is therefore only one-half, instead of an entire, lock full.

going vessels. Besides this idea of a larger canal, there were also traffic and foundation questions that led to the adoption of dimensions and clearances for all bridges such as would conform with a future depth as great as 35 ft. The bridges were intended to carry the heaviest military loads. Based upon these considerations, the characteristics imposed on design were: Depth of foundations of main piers, at least 40 ft. below mean low water; minimum horizontal clearance of navigable span, 175 ft.; minimum overhead clearance for fixed or vertical lift bridges, 140 ft.; floors with bridges closed, approximately at present road levels; and all bridges power-operated.

Surveys and Plans

Surveying for the construction of the sea-level canal was begun immediately after the United States took the project from its former owners on August 13, 1919. The alignment adopted was that laid down by the Board of Engineer Officers convened in compliance with the River and Harbor Act approved March 3, 1909, and charged with making survey and reports for a continuous waterway from Boston, Mass., to Beaufort, N. C.* The task of preparing the requisite working plans and maps included three major surveys: (1) A topographical survey of the entire canal from Delaware River to Elk River, with all adjacent lands for studying the spoil-area problem, and for general information; (2) location of all property line corners to determine whether or not the existing rights of way were of sufficient width, throughout, to accommodate the sea-level canal; and (3) laying out the new canal on the ground, with cross-sections at 100-ft. stations along the center line, for detail work.

Stadia and plane-table methods were used in the topographical surveys and contours were located at 10-ft. intervals. The area covered was about 50 sq. miles and the average width was 2.5 miles. The survey was platted to a scale of 1:9 600, thus requiring three standard 27 by 40-in. sheets for the entire route.

In the next step all property acquired for the new canal was staked out and permanent monuments were placed at controlling and meander corners. The property lines, thus located, were platted on the working drawings to a scale of 1:1 200 so as to show the position of the new canal with respect to the right of way.

The general line of the sea-level canal is substantially the same as the old one, except at the east end, where it strikes the Delaware River about 1¾ miles below the Delaware City lock.

As a part of the layout work, range points for tangents and chords and points of intersection of tangents were monumented. Cross-section profiles were made at each 100-ft. station. Elevations were taken every 10 ft. horizontally at right angles to the center line for a distance of 500 ft. to 800 ft. on each side. Soundings and wye-level measurements were made from the 12-ft. depth in the Delaware River to the Pennsylvania Railroad Bridge, the eastern end of "Deep Cut" Section, and from Guthries Run, the western end of "Deep Cut" Section, to Elk River (see Fig. 1). In "Deep Cut" where the

* H. R. Doc. 391, 62d Cong., 2d Session.

adjacent hills were from 20 to 100 ft. above the canal level, this method was supplemented by hand-leveling, trigonometric leveling, and checking on a wye-level line at the top of the slope along the steep banks.

These data were reduced and the plan was platted to a scale of 1:1 200 which covered 21 standard size sheets 27 by 40 in. The cross-sections were platted to a scale 1 in. equals 100 ft. horizontal and equals 20 ft. vertical. These drawings were the basis of all the necessary designing and computations for the contract specifications and for hired labor work.

Excavation

The canal was divided for excavation purposes into four sections. (See Sections A, B, C, and D, Fig. 1.) Practically all the material to be removed in Section A was under water; hence, it was a dredging proposition. In passing the dredge from the new sea-level entrance to the intermediate level it was important not to permit the water in the embanked part of the old canal to escape. This was prevented by using excavated material to build a dam or dike across the dredged channel, thus leaving the dredge floating in a completely enclosed pond. Water from the canal was then admitted into this pond raising the dredge to canal level, which was retained by the dam. The dredge was then ready to continue on its way into the old canal and complete its work. The operation was reversed in coming out.

Section B contained practically no material above the canal surface, and it was clearly a dredging proposition. The only question to be decided was as to whether a dredge could be introduced into the canal through the locks or by other means.

In Section C (Fig. 1) the height of the banks averaged 50 to 75 ft. above the water of the summit level (17.6 ft.) The excavation was approximately 50% of the total required to complete the 12-ft. canal.

This section of the canal was called the "Deep Cut". It was here that the choice between wet and dry excavation was most difficult to make. The original builders of the canal had piled material from this cut close along its edges, thereby greatly overloading the banks and adding considerably to the material now to be removed for the enlargement. Approximately, 7 000 000 cu. yd. were to be taken from this cut, of which perhaps two-thirds could be removed by dry excavation after first deepening the summit level enough to lower it 10 ft. Even then the remainder below water level would have had to be dredged, and slides would very probably have been induced by this preliminary lowering. The occurrence of such slides in so shallow a channel would have practically closed it to all traffic. Section D was clearly a case for dredging. The combined length of all sections was 20 miles, and the total quantity to be excavated was 16 000 000 cu. yd.

Contracts for Excavation.—The first shovelful of dirt was raised under the new project June 27, 1921, when a preliminary trial contract for steam-shovel excavation was awarded for removing the overburden down to Elevation + 20 (see Table 1, Item 1). The material was transported by three trains of dump-cars to the disposal grounds in Lums Mill Pond which, previous to the acquisition of the canal by the Federal Government, had been used as a feeder

TABLE 1.—DATA RELATING TO QUANTITIES AND COST OF EXCAVATION.

Item No.	Section.	Date operations began.	Date operations ended.	Size of hydraulic dredge intake, in inches.	Material Removed, in Cubic Yards. Gross.	Excess.	Net.	Cost, in dollars, paid to contractor.	Contract unit price, in dollars, per cubic yard.	Remarks.
					Preliminary Trial Contract by Steam Shovel.					
1	C	6-27-21	3-30-23	....	650 924	....	650 924	$ 240 841.88	$0.37	Between Pennsylvania R. R. Bridge and Summit Bridge.
					Contracts for Improvement.					
2 3 4	A	10-15-23	12-30-24	20 23 22	2 419 486	698 184	1 721 302	$ 209 905.22	$0.1244	Three dredges working at different times.
5	A	8- 3-26	3-22-27	22	87 927	31 522	56 405	21 180.80	0.385	
6					2 507 413	729 706	1 777 707	$ 231 086.02		Total for Section A.
7	B	10- 1-23	12- 3-24	22	2 185 076	375 183	1 809 943	$ 247 448.07	$0.139	
8	B	8- 3-26	3-22-27	22	113 086	10 379	102 707	39 027.70	0.385	
9					2 298 162	385 512	1 912 650	$ 286 475.77		Total for Section B.
10 11	C	12-10-22 12-11-23	12-29-24 12-29-24	26 26	5 515 394	59 298	5 456 096	$1 334 230.00	$0.2453	Between Guthries Run and Summit Bridge.
12 13	C	7-21-24 8-25-24	11-10-24 11-15-24	22 26	1 276 952	74 459	1 202 498	274 633.23	0.23	Eastern end of section between Pennsylvania R. R. Bridge and Summit Bridge.
14	C	3- 6-25	5-12-25	26	495 435	43 933	451 502	193 417.01	0.43	
15					7 287 781	177 690	7 110 091	$1 802 280.24		Total for Section C from 1922 to 1925.
16	C	8- 3-26	3-22-27	22	248 569	12 599	235 970	90 630.77	0.385	
17	C	8- -24	11- -24		7 536 350	....	7 346 061	$1 892 911.01		
18				Shovel	56 000	....	56 000	42 000.00	0.75	
19	C				7 592 350	190 289	7 402 061	$1 931 911.01		Total for all of Section C.

TABLE 1.—(*Continued*).

Item No.	Section.	Date operations began.	Date operations ended.	Size of hydraulic dredge intake, in inches.	Material Removed, in Cubic Yards.			Cost, in dollars, paid to contractor.	Contract unit price, in dollars, per cubic yard.	Remarks.
					Gross.	Excess.	Net.			
					Contracts for Improvement.—(*Continued*).					
20		9-24-23	12-31-23	20						
21	D	10-18-23	5-17-24	22	2 858 864	592 062	2 266 802	$ 537 243.13	$0.2447	Elk River Channel.
22		12-12-23	6-11-24	27						
23	D	1- 8-25	5-17-25	26	300 070	51 927	248 143	105 401.82	0.43	Small stretch west of locks at Chesapeake City.
24					3 158 934	643 989	2 514 945	$ 642 644.95		
25	D	8- 3-26	3-22-27	22	50 087	5 450	44 637	16 660.05	0.385	
26					3 209 021	649 439	2 559 582	$ 659 305.00		Total for Section D.
27					16 257 870	1 954 946	14 302 924	$3 352 619.68		Total for all improvement contracts.
					Contracts for Maintenance Work.					
28	A	1926	1927	22	903 792	401 127	502 665	$ 154 997.04	$0.334	Two electric dredges used on this work.
29	B	....	....	....	....	....	....	....	......	
30	C	1926	1927	22	573 169	184 088	389 081	121 270.73	0.334	
31	D	1926	1927	22	565 747	177 083	388 664	124 820.31	0.334	
32		8- 3-26	3-22-27		2 042 708	762 298	1 280 410	$ 401 088.08		Total contracts for maintenance.
					Improvement Dredging with Leased Equipment.					
33	B	1926	1927	Scoop	9 925	....	9 925	$ 7 262.50		
34	C	1926	1927	Scoop	4 725	....	4 725	16 650.00		
35	D	1926	1927	15	31 157	....	31 157	36 442.33		
36					45 807	....	45 807	$ 60 354.83		Total improvement dredging by leased equipment.

TABLE 1.—(*Continued*).

Item No.	Section.	Date operations began.	Date operations ended.	Size of hydraulic dredge intake, in inches.	Material Removed, in Cubic Yards. Gross.	Excess.	Net.	Cost, in dollars, paid to contractor.	Contract unit price, in dollars, per cubic yard.	Remarks.
					Maintenance Dredging with Leased Equipment.					
37	B	5- 1-27	5- 3-27	Scoop	545	....	545	$ 1 500.00		
38	B	2-18-26	7- 2-26	15	216 889	....	216 889	65 347.92		
39	C	3-22-27	6-22-27	22	196 332	....	196 332	113 119.80		
40	D	....	....	....	....	....	....	....		
41					413 766	....	413 766	$ 179 967.72		Total maintenance dredging by leased equipment.
					Improvement Excavation by Government Equipment.					
42	A	7- -24	5- 4-26	Shovel	15 000	....	15 000	$ 28 778.27		Removel of old lock at St. Georges.
43	B	....	....	....	....	....	....	....		
44	C	10- 7-25	2-17-26	Shovel	60 371	....	60 371	39 954.40		Summit Bridge.
45	D	3- -24	10- 5-25	Shovel	49 532	....	49 532	45 491.05		Old lock at Chesapeake City.
					124 903	....	124 903	$ 114 223.72		
					Summary of Total Dredging.					
46	A	10-15-23	1927	....	3 426 205	1 130 833	2 295 372	$ 414 861.33		
47	B	10- 1-23	1927	....	2 525 521	385 512	2 140 009	360 586.19		
48	C	6-27-21	1927	....	9 077 871	374 377	8 703 494	2 466 747.82		
49	D	9-24-23	1927	....	3 855 457	826 522	3 028 935	866 058.69		
50					18 885 054	2 717 244	16 167 810	$4 108 254.03		Grand total for all dredging.

reservoir to supply water to the summit level. The retaining earth dam had been washed out by a freshet August 14, 1919, immediately after the Government had taken over the canal, and the spoil of the operation was used to good advantage in rebuilding the dam, which thus provided a spoil area for material dredged under later contracts.

After this preliminary trial of steam-shovel excavation, dredging was chosen for the remainder of the work. The "Deep Cut" was recognized as the key to the entire job. Therefore, specifications were prepared, after an allotment of \$2 500 000 had been made, with a view to placing a contract for that part of Section C comprising the most difficult work between Guthrie's Run and Summit Bridge. About 5 000 000 cu. yd. of earth were involved in this contract. The specifications were made so as to permit the contractor to use either wet or dry excavation with any combination of methods he might choose.

Operations were begun on December 10, 1922, with a 26-in. hydraulic dredge which had previously entered the canal through the coffer-dam at Chesapeake City. At the same time another 26-in. dredge was brought in; but it was still under construction and did not start operating until one year later, December 11, 1923. (See Table 1, Item 11.) Previous to the beginning of dredging operations a force of men with tractors, dynamite, etc., were at work clearing the north bank of trees, stumps, underbrush, and roots, and constructing a retaining bank and waste weir for the Guthrie's Run spoil area (Fig. 1). The first retaining banks and the waste weir were soon found to be inadequate because of the great quantity of water being pumped with the spoil. This made a more substantial construction necessary.

The method then chosen by the contractor to carry out this work was, briefly, as follows: A steam shovel and the dump-cars from the former contract were used to remove the top of the spoil bank of the old canal. With this material, a heavy earthen embankment varying in height up to 25 ft. was built parallel to the canal and not less than 300 ft. from it, to form one side of the basin into which the spoil was to be placed; usually the other sides were thrown up by a drag-line excavator. As the dredge had an average daily output of about 7 000 cu. yd. on this work, cross-dikes were run, as needed, to provide smaller basins into which it could discharge without interfering with the construction of the main dike farther along. The contractor purchased about 500 acres of land for this purpose which, added to approximately 300 acres owned by the United States, made a total of about 800 acres devoted to these spoil basins. The embankment on the canal side was about 24 000 ft. long. The waste water from the entire area was forced by a series of levees, weirs, and ditches to settling ponds north of a public road paralleling the canal. After a travel of 2 to 3 miles through these basins, the water returned to the canal quite clear. This return was quite essential because all the water in the canal had to be supplied by the Chesapeake City pumping plant, the capacity of which would not have been adequate otherwise.

The material encountered in this section consisted of strata of loam, sand, some gravel, hard blue mud, and clay. The hard blue mud and the clay could

not be broken up, but came through the pipe line in slices and lumps, piling up at the end of the pipe line without running off.

There was little delay caused from choking of the pipe line, although the lines were run up almost perpendicular banks from 60 to 80 ft. high. The greatest height over which material was discharged was 97.5 ft. above the water level in the canal. The major part of the rip-rap stone revetment of the old canal along the old two-path was removed by a small bucket dredge, before the dredging began.

The only damage done to plant in boldly attacking banks of maximum height (aside from normal deterioration) was an occasional broken spud or hoisting cable. The character of material was such that instead of flowing to the intake pipe of the dredge under a gradual slope, large masses of it would either slide or break off and fall into the water without warning; this caused waves which would occasionally break the spuds before they could be hoisted. No structural damage was done to the plant. The difficulty was partly overcome by washing the banks down by flowing water or jets; but the most successful method was by the use of blasting powder, first drilling a series of holes along the required slope line to depths varying from 10 to 25 ft. and about 20 ft. apart. The explosion of this powder would usually cause the earth to fall into the head of the cutting in a more gradual manner.

The part of Section C from the Pennsylvania Railroad Bridge to Summit Bridge was awarded in two separate contracts. On July 21, 1924, a 22-in. hydraulic dredge started at a bend in the old canal 1 600 ft. west of the Pennsylvania Railroad Bridge and worked east, leaving the old canal to the north (Fig. 1). Since the only available spoil area was Lums Mill Pond, it was necessary to lay a submerged pipe line across the old canal to dispose of the dredged material without interrupting traffic. On August 25, 1924, another dredge started at the same bend and worked west, spoiling in the same area through a pipe line from 1 000 to 4 000 ft. long with the discharge at Elevation $+42.5$ ft. The total quantities excavated on this contract are given in Table 1 (Items 12 and 13).

Operations were begun on a new contract on March 6, 1925, by a dredge, working west from a point 1 200 ft. east of Summit Bridge (Fig. 1) where the overburden had previously been removed to Elevation $+20$ by the steam shovel. An area just east of the public road leading from Summit Bridge, Del., to Glasgow, Del., purchased for spoiling purposes, was enclosed by 6 000 ft. of levee. (See Table 1, Item 14.)

Section B is about 3 miles in length, extending from St. Georges to the Pennsylvania Railroad Bridge. A 22-in. hydraulic dredge entered the canal through the coffer-dam at Chesapeake City, proceeded east to St. Georges, and began operations on October 1, 1923. (Table 1, Item 7.) The course of the old canal was followed westward from St. Georges to Lorewood Grove. At this point the tow-path was cut through and the new canal diverged gradually to the north to eliminate bends. The greatest divergence was approximately 300 ft. passing through Otter Island and a point of high land with an elevation of $+37$ ft. on the north, then crossing the old canal to the south of the

Pennsylvania Railroad Bridge. This provided an improved approach to the new bridge to be constructed for the Pennsylvania Railroad Company.

This course necessitated the clearing of some woodland by removing the trees and stumps with dynamite and tractors, and grubbing the underbrush and roots ahead of the dredge. The dredged material consisted of sand, clay, and gravel and was deposited with little difficulty in the coves and behind the old tow-path. A small dipper dredge was used to remove the rip-rap stone from the old tow-path in the line of the new canal and to construct approximately 2 500 ft. of retaining bank.

Dredging operations on Section A began at Reedy Point, Del., with a 23-in. hydraulic dredge, working westward through the low marshland and the upland to the Delaware City-Port Penn Road. At this point operations were suspended on December 16, 1924, to prevent the flooding of several hundred acres of low land lying to the south. After resuming dredging operations at the Delaware City-Port Penn Road on July 7, 1925, the contractor's dredge continued westward to join the old canal at "Wide Waters." The mud, sand, and gravel dredged up to the road was deposited in spoil areas on the north side of the canal. Two of these were low, swampy places on the Fort du Pont Reservation, which required the construction of approximately 3 800 ft. of pipe line. The only difficulty encountered was caused by the sand that settled behind stones lodged in the line. The bulk of material dredged from the canal prism in this stretch was deposited along the face of the dike as a reinforcement and protection against erosion from tides and wind.

Before a dredge could enter the old canal at "Wide Waters" it was necessary to construct a levee behind it, 600 ft. long, with the top at Elevation + 14.0, from the high land on the south to the old canal bank on the north. The water from the canal was let into the basin, thus raising the dredge to canal level, at which it then continued operations westward to St. Georges. The material dredged through "Wide Waters" was principally vegetable matter, which would rise to the surface of the water in lumps and float off like small islands. A mixture of mud, sand, clay, and gravel that impeded progress was encountered near St. Georges. The material was deposited in the coves along the south side of the canal and on the low lands to the north.

Three hydraulic dredges were in operation on this contract at different times; one of them did 45% of the work and completed the contract on December 30, 1924. It passed out of the canal the same way that it entered. The quantities dredged under this contract are listed in Table 1 (Items 2, 3, and 4).

Section D was awarded in two separate contracts (see Table 1, Items 18, 19, 20 and 21). That of the Atlantic, Gulf, and Pacific Company covered the whole section except a small stretch from a point just west of the locks at Chesapeake City. A total of 830 700 cu. yd. of mud, sand, clay, gravel, and some sandstone was excavated from the 4¼ miles of channel from Elk River to Chesapeake City. It was deposited in the coves on both sides of the channel without difficulty other than that usually attending dredging operations. The channel was completed to Chesapeake City on December 31, 1923.

Two dredges were admitted into the canal separately through the cofferdam at Chesapeake City and worked east along the lines of the old canal to

Guthries Run, a distance of 2½ miles. From a point 600 ft. east of the old canal waste-gates at Chesapeake City to the Maryland and Delaware line the material consisted of a tough red clay with a small percentage of sand and gravel, which was easily disposed of in the coves to the south and behind the old tow-path on the north side. From the Maryland and Delaware line east to Guthries Run the land rises from the canal level to Elevation +45. The contractors purchased two tracts of land for disposal areas, one on each side of the canal. To enclose these basins required approximately 12 000 ft. of levees which were constructed with dredged material in a series of steps from canal level to Elevation +50. The material was mostly sand, loam, and gravel, with a small percentage of red clay. Considerable difficulty was encountered in maintaining the levees, and several washouts that released thousands of cubic yards of sand, made redredging necessary; but this did not impede traffic in the canal to any great extent.

On January 8, 1925, a 26-in. dredge began work 600 ft. east of the waste-gates, which was extended west to the old drawbridge at Chesapeake City, a distance of 1 800 ft. On February 28, 1925, work was suspended and the dredge was moved to Section C until May 12. Work was resumed on May 13, and the contract was completed May 17, 1925. The material, consisting of red clay and sand, was deposited in Broad Creek spoil area.

There were eight separate contracts for excavation awarded from June, 1921, to March, 1925, four to W. H. Gahagan, Incorporated, two to the Arundel Corporation, and two to the Atlantic, Gulf and Pacific Company. The plant in operation consisted of one steam shovel, seven powerful hydraulic dredges, two bucket dredges, two scoop dredges, two dragline banking machines, and a number of small gasoline tugboats used for transporting men and towing fuel-oil barges, etc., to and from the various plants. Each dredging company proved its efficiency to cope with such a large undertaking in the smooth and satisfactory manner in which every obstacle encountered was met and they completed all contracts in advance of the specified time. The total quantities removed from the canal prism under these contracts were 16 202 000 cu. yd. of material at a cost of $3 311 000, or, approximately, 20 cents per cu. yd.

Excavation work was suspended after May 17, 1925, to await the completion of the highway bridges. Owing to a delay in the construction program there were four bridge sites at which no excavation was done under contract: St. Georges, 700 ft.; Pennsylvania Railroad Bridge, 500 ft.; Summit Bridge, 500 ft.; and Chesapeake City, 1 200 ft.; or, a total of 2 900 ft. of channel.

Only one man lost his life on all excavation contracts. He slipped from the pontoon line of one of the dredges and was drowned. The damage to plant other than broken spuds and the usual wear and tear attending dredging operations was slight. The regular boat traffic through the canal was not interrupted to any great extent except when the dredges were being taken in and out of the canal through the coffer-dam at Chesapeake City, which required two or three days for each operation.

In the meantime, Government plant began the removal of the old original locks at St. Georges and Chesapeake City, constructed between 1824 and 1829. A dam of timber and steel sheet-piling was first constructed at Chesapeake City

to retain the canal level so that the lock could be kept in operation and yet all dredging of the channel around it could be completed except at the dam.

Leased Plant.—By January, 1926, sufficient progress had been made on the bridges and the removal of the old locks to resume dredging operations. A 15-in. hydraulic dredge and its attending plant was leased at $700 per day. It was the only hydraulic dredge available that could pass through the canal locks. Accordingly, it began work at Chesapeake City, where the old locks had been located, and dredged to project dimensions (Fig. 1). It was then moved to Summit Bridge, but could not pump the material to the elevation of the spoil area. Next, it was moved to the Pennsylvania Railroad Bridge and worked from that point to St. Georges, doing maintenance dredging until June 30, when it was released. It removed about 34 700 cu. yd. of mud, sand, gravel, and clay, under improvement and 217 000 cu. yd. under maintenance, at a total cost of $106 000.

When it was found that the 15-in. dredge could not complete the reconstruction of the canal and that a more powerful one would be required, negotiations were started for a 26-in. dredge to be leased for 60 days for $169 000 and at $1 500 per day thereafter. The dredge was available immediately and was started from New York City on March 30, 1926. It foundered en route off the New Jersey Coast during a storm on March 31, and was a total loss.

Final Contract.—After various unavoidable occurrences had impeded the progress of the work, plans and specifications were prepared to complete the work by contract. Bids were invited and the contract was awarded to the Atlantic, Gulf and Pacific Company, at 38.5 cents per cu. yd. for improvement and 33.4 cents per cu. yd. for maintenance dredging.

It was necessary to construct a coffer-dam at St. Georges to make a basin 190 ft. long and 140 ft. wide to raise the dredge to the upper canal level (Elevation +17.6) without interfering with traffic in the canal. Work was started about July 15, 1926, and was completed August 24. The old lock wall was used as the south side of the coffer-dam. The east side consisted of two rows of 35-ft. steel sheet-piling 15 ft. apart. These were tied together on the inside by cross-rows of steel piles spaced 20 ft. apart, thus forming pockets that were filled with earth. An opening for the dredge to enter was provided in the east end of the coffer-dam.

The north side was of the same construction with the two rows of piles, 14 ft. apart and from 14 to 35 ft. long, extending for a distance of about 70 ft. to high banks. Then a dirt bank, with wooden piles and sheathing as a core, extended up the side of the bank to Elevation +20.0. The west end of the coffer-dam was the old roadbed.

A 20-in. hydraulic dredge began operations on August 3, 1926, pumping a dam across the new entrance to the canal 15 200 ft. from Delaware River. This formed a basin by which to raise the dredge from tide level to canal level at Elevation +7.6. The dredge then cut through a dam constructed under a previous contract, entered the canal at "Wide Waters", and continued westward on maintenance dredging to St. Georges, entering the coffer-dam on August 25.

The time required to pass the dredge through the coffer-dam to the upper level was five days. Leaving St. Georges, it proceeded west to the Pennsylvania Railroad Bridge and began operations on improvement dredging at that point. It was engaged there until September 27, when it was found that the bridge piers were settling before project dimensions had been obtained in the draw-opening. The dredge was withdrawn and placed on maintenance work between the railroad bridge and Summit Bridge until October 11, while steps were taken to restore the bridge to condition. From October 12 to November 13, 1926, improvement dredging was completed at Summit Bridge, and the dredge continued westward on maintenance work until January 24, 1927.

Canal Lowered to Sea Level.—On January 25, 1927, the dredge was moved to Chesapeake City to dredge up to the retaining dam in preparation for lowering the water in the canal to sea level. The wickets were opened in the locks at Chesapeake City and Delaware City, on February 1, 1927, and, as the make-up water escaped, the dredge continued to work through the old roadbed. It reached the retaining dam on February 7, and was then moved back to complete the maintenance dredging between Chesapeake City and Bethel while a force of men removed the retaining dam. This was practically completed by February 20; dredging was resumed at that point on February 22 and completed on February 28, except a part of an old lock wall that could not be removed by the hydraulic dredge.

When the canal surface reached sea level a great number of small slides occurred along the banks of the canal, which usually did little or no damage to the channel. At a point 3 500 ft. west of Summit Bridge a slide occurred on the north side about 600 ft. long. It moved gradually into the channel, forming a shoal of the same length; 50 083 cu. yd. of material were removed from this slide by dredging from March 1 to March 22. The total net yardage removed under contract by this dredge was 1 211 000, of which 352 000 was for improvement and 859 000 for maintenance.

A 30-in. hydraulic dredge arrived at the Delaware River entrance of the canal on December 22, 1926, and began dredging on maintenance work at the 12.0-ft. contour in Delaware River. It worked west for a distance of 15 200 ft. and began removing the dam constructed by the first dredge at the same time the first dredge was removing the one at Chesapeake City.

The removal of this dam required about three days and the material was deposited in the Delaware City branch of the canal, thus forming another dam to prevent the tide-water from entering by way of that channel. The dredge was moved to St. Georges on February 10, 1927, for improvement dredging at that point, but was delayed to some extent because of the difficulty encountered in removing the steel piles from the coffer-dam. The removal of these and the great amount of timber and débris in the old roadbed required eleven days. The dredge was then moved to the Pennsylvania Railroad Bridge (February 23) and removed the last obstruction to navigation in the canal, thus making it possiblé to pass through at sea level for the first time.

In addition to dredging done under contract at the railroad bridge, 12 000 cu. yd. of over-burden were washed from back of the piers into the canal

to relieve the back pressure; this was later re-dredged. The total net yardage removed under contract by this dredge was 509 000, of which, 87 300 was for improvement and 421 700 for maintenance.

The entire contract was completed on March 22, 1927, after a net total of 1 720 000 cu. yd. of material had been removed and re-deposited in the same spoil areas that had been used under previous contracts. The cost was $568 600, of which $167 500 was for improvement and $401 100 for maintenance.

Cleaning Up.—There were two places in the canal that could not be dredged by the hydraulic method, a bed of boulders 150 ft. long and 50 ft. wide on the south side of the channel at Goose Point, Del., and a part of the old lock at Chesapeake City. For this work, a dipper dredge was leased for a period of 60 days at $500 per day. No great difficulty was experienced in removing the boulders, although some weighed as much as 7 tons. These were loaded on the decks of scows and placed on shore by derricks. In addition to the removal of the boulders and lock wall, several small slides were dredged from the channel, loaded in scows and dumped in front of a 22-in. hydraulic dredge to be rehandled. The total for this item was 11 600 cu. yd. of material (largely rock) removed at a total cost of $32 300.

On March 22, 1927, a slide occurred on the south side of the canal at Guthries Run and a 20-in. dredge was leased for 90 days at $1 500 per day for the purpose of removing this and any other slides that might develop. While the removal of this slide was in progress, the one previously removed and another just west of Summit Bridge were gradually moving channelward and had to be re-dredged.

The canal was completed to project dimensions on May 31, 1927, although it had been provisionally opened to commercial traffic on February 25, 1927. During this time its use had been restricted to daylight periods of high water and to boats of 9-ft. draft and vertical clearance of 45 ft. The canal was formally opened on May 14, 1927. The total quantities and costs involved on all kinds of dredging work is given in Table 1.

Land Slides

Old records indicate that when the canal was originally cut through the summit divide, and for several years thereafter, much trouble was experienced from sliding banks in that particular section, especially in the first mile west of Summit Bridge. (See Fig. 1.) The slides finally became quiescent and remained so until they were again set in motion by the operations of reconstruction. Fig. 2 is a view of the original canal. Sliding was undoubtedly caused partly by the practice of the original builders of disposing the spoil very close to the edge of the cut. In some places this spoil was piled up so as to raise the height of the bank from 100 to 110 ft. above sea level; that is, 20 to 30 ft. above the original level. This additional burden undoubtedly contributed to sliding, but the primary cause was the slow deformation of the clay subsoil.

Borings made along the entire length of the canal showed that beneath a top layer of green sand impregnated with some mica, there were various combi-

FIG. 2.—PART OF ORIGINAL CANAL IN SECTION C, WHERE SLIDES OCCURRED.

FIG. 3.—VIEW OF SLIDING BANKS IN SECTION C.

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nations of sand and mud, all of which indicated the decomposition of a heterogeneous collection of rock. Pockets of vari-colored clays were apparently interspersed with similar sands. The results of some borings were interpreted to indicate the presence of quicksand. A Board of Engineers expressed the view* (based on the results of borings) that some slips due to water at the upper surface of some of the clay strata, as well as some due to water-saturated material might be expected during and after the proposed re-construction. It was fully expected, therefore, that there would be some trouble from sliding banks during and after reconstruction and that they might, at times, interfere somewhat with navigation. The Board believed, however, that slides could readily be handled by following a policy later adopted on the Panama Canal; namely, to provide means of removing them as they developed until the banks became quiescent. A few minor slides did develop during the dredging operations, but owing to the precaution taken to maintain the canal at the high level during the construction period, no damage was done, and traffic was not obstructed by them.

Serious trouble did not develop until after the canal had been lowered to sea level, and again opened to traffic. About ten days after the surface level had been lowered, a section of the south bank near Mile 9 suddenly gave way and partly closed the channel. This was promptly removed by the dredge which had just completed improvement work. In the meantime, several slides were observed to be in motion within the region of the old slides in the first mile west of Summit Bridge, and close enough to the new bridge to cause some anxiety. These moved very slowly, and gave time for a careful examination to be made of their progress and effects. Usually, they were manifest first by the appearance of cracks on the surface of the ground at a distance of 15 to 20 ft. back of the edge of the bank, and parallel with it. The material between the crack and the edge of the bank would then sink almost vertically, the rear portion somewhat more rapidly than the front, so that the original ground surface of this slice of moving material would slope away from the canal. At the same time that this settlement was taking place, there was a general upheaval and breaking up of the material at and near the toe of the slope, extending 15 to 20 ft. above the water surface and encroaching slowly on the channel. The general effect is shown in Fig. 3. After one slice of the banks had settled 8 to 10 ft., a similar crack would appear back of the old one and a new settlement would begin. A series of three or four slices would present the form of a giant stairway.

In addition to the encroachment on the channel laterally, there was also an upheaval of the bottom of the canal, so that both boat traffic and tidal flow would soon be seriously hampered unless dredges were promptly put in operation to remove the invading material. Boring samples were found to be unreliable when checked against the material actually encountered. Excavation showed the material comprising the canal prism in the deep cut to be of three general classes. From the surface down to about Elevation +50 the material was largely a yellowish, loamy sand containing much ferruginous

* H. R. Doc. No. 391, 62d Cong., 2d Session, pp. 91-93.

matter. It was interspersed with many layers of yellow or reddish iron ore, forming hard concretions. The lower portion of this sand was water-bearing.

Below this, extending from Elevation +50 to about Elevation +5, there was found a compact blue mud, largely of argillaceous material, but containing some siliceous matter. This mud was quite compact, and, although damp, was practically impervious to water. On weathering, it disintegrated into a fine, grayish dust. It is evidently material laid down in a previous era of submergence under fresh water, since it contains no marine fossils.

Immediately below this mud, and showing at about high-tide level, there is a thin layer of lignite which rests on a thick stratum of clay. This underlying bed of clay is moist and plastic and is composed of almost pure kaolin, usually of a whitish or grayish color, but occasionally colored red. Lying 80 ft. below the surface of the ground, it is under considerable pressure. The excavation of the canal prism naturally disturbed the distribution of pressure in the vicinity of the cutting. Being plastic, this material readily changes shape to conform to the new distribution of pressure, and it is this slow deformation of the clay stratum which gives rise to the slides.

No method of preventing the slides is known; drainage would have no effect. It is quite probable that the banks will eventually reach a state of equilibrium as they did formerly; in the meantime, however, much expense will be involved in their removal, and navigation will experience the hazard of occasional interruption. It is fortunate that the section most likely to slide is comparatively short and does not involve the entire deep cut.

Jetties

After initiating the excavation work, the next step was the construction of jetties at the outlet of the new entrance in Delaware River. The bottom of the river at this point is very soft mud of considerable depth, apparently about 35 ft. The tidal currents flowing back and forth across the front of this entrance would wash the soft material into and quickly fill a dredged cut if it were not prolonged into deep water by confinement between jetties, which concentrate the canal flow and protect the canal cut from the river currents. Studies were made to determine the character of the bottom, and of various types of jetty suitable for the very soft bottom encountered. The decision was to use a rubble-mound stone jetty founded upon a brush mattress. Since brush mattress construction is not well known among contractors in this vicinity, and since the structure was expected to settle an indeterminate distance into the soft mud, it was decided to construct these jetties by hired labor, the brush and stone being purchased by contract.

The two jetties are each approximately 1 350 ft. long and extend from the shore line into a depth of about 25 ft. of water. They are spaced 500 ft. apart at the shore end and 800 ft. apart at the outer end. The tops of the jetties are 10 ft. wide and are at a height of 10 ft. above mean low water.

The brush mattresses extending the entire length of the jetties are 2 ft. thick and 60 to 80 ft. wide. They were sunk as built by a 2-ft. layer of small stone placed upon them, and upon this small stone was placed the larger stone forming the body of the jetty. The large stone varied in size from 10-ton

blocks to "one-man" sizes. These jetties, which were completed April 1, 1926, have not yet (November, 1929) shown any signs of settlement. This is an indication of the great efficiency of the brush mattress as a support. The cost of the jetties was approximately $350 000.

St. Georges Creek Tide Dike

The eastern part of the canal between the Pennsylvania Railroad Bridge and a point about 1 mile west of Delaware City occupies the bed of old St. Georges Creek. The new entrance to the canal, starting at Reedy Point on the Delaware River, follows a straight line running westerly in direction until it intersects the bed of the old creek near the former line of the canal at a point about 1½ miles south of Delaware City. In its natural condition, St. Georges Creek extended through a wide marsh area, much of which has been incorporated in the right of way for the canal.

This marsh area was protected from inundation by tides from the Delaware River by levees along the river banks. The opening of the new entrance channel of the canal would have had the effect of admitting the tide into this protected area; hence, in connection with studies for the details of the project, the necessity for modifying it so as to provide protection for the marshes became apparent at an early date. Furthermore, a county road running approximately parallel with the old canal and crossing the marshes (the Dutch Neck Road), was to be crossed by the new entrance, yet this road was not deemed sufficiently important to justify providing a separate bridge. It was decided to provide for these necessities by the construction of a light earthen dike parallel to the new canal on its south side, which would shut the tides out from the remainder of the marshland on the south and would carry the Dutch Neck Road to the Delaware City-Port Penn Road, enabling one bridge to serve both. The construction of this dike had not been provided for in the project document, and, therefore, its cost had not been included in the estimates.

A study was made of the most suitable methods of constructing the proposed dike, and two were considered practicable and economical. It was expected that the dredging contractor might arrange with marshland owners to use some of it for spoil, in which case the spoil itself could be used to construct the bank, but such arrangements could not be consummated. It was then decided to attempt to get a bucket dredge into the marshes for the purpose of constructing the dike by throwing up a suitable embankment, but as the marsh had no outlet by which a dredge could enter, it appeared necessary to construct the new canal entrance up to a point where a supply of tide water could be introduced, and this maneuver was successfully accomplished, with but little effect on the marshlands. The dredge cut its way in through a small narrow channel, which it filled behind itself. The construction of the dike was then conducted under an emergency contract made under proper authority.

This contract was at $3.90 per lin. ft. of completed dike. Work was begun on February 8, 1924, by a bucket dredge which had been pulled over the canal bank and had dug its way down St. Georges Creek to the site of the work.

Progress was slow, however, because the material was mostly vegetable matter and compacted badly. A second bucket dredge was put on the work on May 10, and, by the end of June, the dike was deemed of sufficient height to protect the meadow under ordinary tidal conditions. Operations were then resumed on the new canal on July 7, allowing the tide water from the Delaware River to flow into the low land between the dike and the old canal. On July 16 an extremely high tide topped the dike and caused two breaches which flooded the low land to the south. The bucket dredge continued working on the weak places, but flooding conditions continued to grow worse until September 27, when the Federal Government took over the work of completing the dike. Unfortunately, the material comprising the marshes had proved utterly unfit to support the dike against the pressure of the tides; the weight of the tide water consolidated the bed of the marshes, causing it and the dike to settle so rapidly that this, combined with heavy rains and high tides, soon caused breaches.

Prior to beginning this work it was known that the material comprising the marsh was of a peaty nature of considerable depth, but it was believed that there was a sufficient quantity of mud in it to make it form a substantial dike. Subsequent experience has shown that such was not the case. Moreover, it was exceedingly improbable that any previously projected dike would have been successful, because, lacking previous experience with this material, the design would have been faulty.

The peat comprising the body of this marsh is a spongy vegetable growth, bound together by a matted mass of roots. When a piece of it is separated from its natural submerged bed, it floats. Large chunks occasionally come to the surface and float away. It is also very compressible and acts in a manner similar to rubber. These characteristics made it impossible to build a dike from the dredged material, and better fill had to be brought from the outside. As soon as this heavier material was placed on the marsh it compressed and settled, until, finally, it acted like rubber or a compressed spring. It buckled laterally under the weight and forced the surrounding material up. This action would continue until the entire quantity of the lighter material in a vertical section had been completely replaced by the heavier material.

If these difficulties could have been foreseen before introducing a dredge into the marsh, the heavy material would have been brought in by carts or by narrow-gauge trains. Both these methods were used in subsequent operations, thus supplying data on which to base an opinion of the difficulties that would have attended the construction from the beginning of the entire dike by either method. Carts were fairly successful, but too slow, because enough of them could not dump at the end of the fill to make rapid progress. When parts of the dike turned over, or moved laterally (settling at the same time) no damage was done when carts were used. The use of narrow-gauge trains was not possible until enough solid material had been put in place by other methods to keep the track from sinking, and wrecking the plant.

When the first breaches occurred in the dike, attempts were made to close them by sand-bags. These bags were usually successful in preventing a breach

from widening or deepening, but were never successful in effecting a complete closure.

The next attempt at closure was to build the remaining sections of the dike up to a grade safely above high tides by adding sand and other heavy material through bleeders in the pipe lines of the dredges. Then, after all unharmed parts of the dike were thus strengthened and reinforced, the discharge was directed so as to build from the two ends, at the same time pumping as much heavy material as possible into the breach. Two large breaches were finally successfully closed in this manner, but even then the dike continued to settle, and soon became in danger again of being overtopped by high tides. A steam shovel was then procured, with 36-in. gauge locomotives, necessary dump cars, track, etc., and the work of hauling solid material by train from a near-by hill was begun. The dike has now been filled to Elevation +14, and is 18 ft. wide on top. The cost, including attempts to close breaches, was about $300 000.

Highway Bridges

The re-construction of the canal required changes in six bridges, one carrying railway traffic and five highway traffic. An additional structure was required over the new entrance south of Delaware City, now called the Reedy Point Bridge, but originally designated the New Cut-Off Bridge. There were wooden jack-knife swing bridges at four of the points, and those at Chesapeake City, Summit Bridge, and Saint Georges were on highways vital to traffic on the peninsula. The Summit Bridge was an iron horizontal swing drawbridge. By arrangement with the State of Maryland it was decided not to replace the structure at Bethel known as Maryland Pivot which was removed as soon as dredging operations made it necessary. Bids for a new bridge over the part of the old channel in Delaware City proper were deemed unduly high and it was decided not to construct a new bridge there, since traffic did not require it, but to alter the foundations of the existing structure at that point and to strengthen it sufficiently for present needs.

The new bridges were designed to serve the canal, not only as constructed under the present project (that is, at sea level with a 12-ft. depth and 90-ft. bottom width), but also as projected for the future, dredged to tide-water with a depth of possibly 30 ft. and a bottom width of 150 to 175 ft., with provision for a horizontal clearance of 175 ft. and a vertical clearance of 140 ft.

The vertical lift type of bridge was selected for all locations on the basis of lower first cost and maintenance expense. Furthermore, fireproof floors could thus be provided economically, and most of the vessels would require the bridge to be only partly open, thus reducing further the operating expenses and delays to land traffic. Operation is by direct current from storage batteries that are charged by a motor generator set using commercial alternating current; emergency operation is by gasoline engine which can either drive the generator or raise the span by direct connection through a belt to the gear train.

All bridges are of the through truss type and were designed to accommodate highway traffic adequately for future years with the continued growth

of motor transportation, and to carry safely loads incidental to troop movements. The clearance diagram and design loads for the bridges at Chesapeake City, Summit, and St. Georges, are shown in Fig. 4. Sidewalks 4 ft. 6 in. wide were provided outside the trusses. The roadway clearance of the Reedy Point Bridge is 4 ft. narrower than that of the other three; otherwise, the clearances are the same.

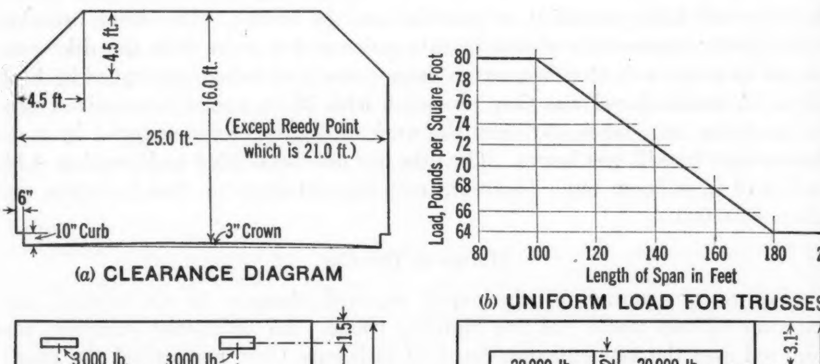

FIG. 4.—LOADING DIAGRAMS AND CLEARANCES FOR BRIDGES AT CHESAPEAKE CITY, SUMMIT, AND ST. GEORGES, CHESAPEAKE AND DELAWARE CANAL.

Loading.—The live loads used for design were as follows:

For the floor system of the Chesapeake City, Saint Georges, and Summit Bridges, two 15-ton trucks abreast, or uniform load, as shown in Fig. 4(*a*), or one 40-ton tank.

For the floor system of the Reedy Point Bridge, two 15-ton trucks abreast, or uniform load as shown in Fig. 4(*b*).

For all sidewalks, 100 lb. per sq. ft.

For all trusses, uniform live load, as shown in Fig. 4, over the entire roadway plus one-half this uniform load on the sidewalks, or the loadings specified for the floor system.

An impact allowance of 30% was made for all live loads.

Canal traffic was required to be maintained uninterrupted during the construction of any bridge; hence, the specifications called for a vertical clearance of 55 ft. for a width of 25 ft. to be maintained in all falsework over the channel.

Since the ultimate canal will have a depth below low water of possibly 35 ft. and since no rock was found in any exploration borings, the least depth to which pier bases might safely be carried was considered to be Elevation —40. The plans in each case stipulated that caissons be sunk either by

open dredging or by pneumatic process at the option of the contractor, and that in either case they must be unwatered for examination of the foundation material before sealing. The contractor elected to use a caisson of his own design on all contracts and to sink it by open dredging until such time as this method became impracticable, and then to resort to the pneumatic method.

Except at Summit, where the so-called abutments are really buried piers, the abutments are of conventional type with wing-walls parallel to the center line of the roadway and supported on piling.

Operating Machinery.—These vertical lift bridges are simple spans suspended at each corner by wire ropes which pass over sheaves on towers and connect to counterweights about equal to the weight of the span. The span is arranged to move up and down vertically in such a manner as to be always parallel to its lowest position. The counterweights move in a similar manner inside the towers. The operating machinery comprises four spirally grooved drums connected through trains of gears to a direct-current electric motor (or gasoline engine auxiliary unit), all of which are mounted on a housed-in platform at the center of the lift span and just above the roadway clearance. Each drum is wound with and controls two pairs of operating ropes which lead from them horizontally to a corner of the span, pass either over or under deflector sheaves, and thence, vertically, one upward the other downward, to connections on the tower columns. Revolution of the drums in one direction winds on all up-haul ropes, "pays off" all down-haul ropes, and raises the span. Reversal of the direction of rotation of the drums pulls the span down.

The operating ropes connect to take-up devices for adjustment of length and tension. The electric motor is governed by a drum controller and is equipped with a solenoid brake, with a hand-brake as auxiliary equipment. Thus, the span is moved, held level during operation, and may be stopped and held at any point by stopping the machinery.

When a span is raised, say, 90 ft., there is transferred from the span side of the tower sheaves to the counterweight side, 90 lin. ft. of suspending ropes at each corner of the span. This is no inconsequential weight and would materially affect the balance of the span were it not compensated. For the short lift at Summit (68 ft.) this change in balance is not dangerous; nor does it cause excessive power consumption. Compensation is obtained on the other three bridges by balance chains hung between each counterweight and its tower so that as the span rises and is lightened by the loss of suspending ropes on its side of the sheave, double this weight of the chains is removed from the counterweights. The balance thus remains unchanged for any position of the span.

Use of Concrete.—Each counterweight is a block of concrete with pockets or wells at the top for balance blocks. A structural steel girder is used to carry the weight of the forms and the first, or girder, layer of the counterweight itself. This girder is a heavily reinforced concrete beam designed to carry the superimposed concrete of the completed counterweight and, therefore, it was allowed to cure thoroughly before any further concreting was done on the counterweight. The forms used were of $\frac{5}{16}$-in. and $\frac{9}{16}$-in. steel plates held to proper width and length by $\frac{5}{8}$-in. tie-rods. Difficulty was experienced

in obtaining plane faces because no wales or continuous stiffeners were used and the sheets were too thin to have the necessary stiffness.

Materials and Concreting.—All materials were delivered by barge. Sand and gravel were stored on plank platforms. Water for the concrete was taken from the upper level of the canal for all jobs except that at Reedy Point, where wells were sunk to avoid the use of brackish tide water. The specifications called for the concrete to be proportioned by volume of materials, but to contain not less then 6 sacks of cement for 1 yd. of 1:2:4 concrete, nor 5 sacks for 1:2.5:5 concrete.

Some variation occurred in the gravel and there were natural variations in the sand due to moisture content. It was the practice to vary the proportions of the aggregates for workability and to keep within a 0.1 sack of cement of the prescribed minimum, always keeping the water ratio as low as possible. Except at Summit, measurement was by wheel-barrow. However, by calibrating the bucket into which the mixer discharged, a constant check was available on the cement content. After each day's run the sack count was checked against the concrete yardage placed in the forms. At Summit Bridge, conditions permitted two whirlers to operate within reach of one mixer, and a batcher was built which permitted the use of a method that proved superior to the antiquated one used on the other jobs; one whirler handled concrete and the other fed the batcher. Water regulation and control were rigidly enforced, and the concrete never showed a slump test exceeding 2 in.

When the temperature was below 40° Fahr., the water was heated by turning the mixer exhaust into the tank and steam jets were placed in the aggregate piles. Fresh mass concrete was covered by tarpaulins and heated by steam. All materials were unloaded to stock by whirlers which also handled the concrete between the mixer and the forms.

Tower Straightening.—In bridges of this type the entire load of the span and counterweights is carried by the front legs of the towers, the rear legs acting only as bracing members to carry dead and wind loads. When the load is applied to the front legs or columns of the towers, there is an appreciable shortening due to the elasticity of the metal; consequently, these legs are fabricated longer than normal in order to compensate for this shortening under load. As this shortening takes place there is a progressive forward rotation of the tower about each rear-leg panel-point, which extends to the bottom. In order to have these tower columns plumb in the completed bridge they are erected in a cambered position. In order that the span will hang free of its guides and thus operate with a minimum power demand, the towers must be plumb in the transverse direction. Furthermore, both columns of the tower at the fixed end of the lift span must be in the same vertical plane to prevent binding of the jaw-guides which hold the span against longitudinal movement. The heights of the towers from the top of the masonry to the center line of the sheaves and the extreme span lifts are as given in Table 2.

The towers of the Saint Georges and Chesapeake City Bridges were plumbed by using plumb-bobs and those of the Summit and Reedy Point Bridges, by transit. Each method has its advantages and either will give accurate results under proper working conditions.

Electrical Equipment.—The electrical units installed on the various jobs are as listed in Table 3.

Illumination.—All illumination is by electric power from the storage battery. There are lights in the battery house and in the operating room, on the stairs to the operating room, on the towers at the foot and top, for roadway illumination, as required by law for navigation, warning signals on the barrier gates, and flashing signals at the portals. As these lights are on wiring circuits regularly carrying the normal battery voltage and as the voltage is much higher during the time when the battery is being charged, variable resistances are installed in all lighting circuits to permit reducing the voltage to a safe maximum.

TABLE 2.

Bridge.	Height of tower, in feet.	Height of lift, in feet.
St. Georges, Del.	168.1	123
Chesapeake City, Md.	186.0	131
Summit Bridge, Del.	115.6	71
Reedy Point, Del.	174.1	131

Telephone.—All bridge operating rooms are connected by telephone to commercial exchanges regardless of the position of the span. This is accomplished by means of a flexible cable between the span and one tower. The suspension point on the tower is slightly above the mid-height. A small sheave riding in the loop of the cable and held by a vertical guide-wire restrains the cable from blowing against the side of the tower, or under the span.

TABLE 3.—List of Electrical Units at the Various Bridges.

Units.	St. Georges, Del.	Chesapeake City, Md.	Summit, Del.	Reedy Point. Del.
Total moving load, in tons.	1 425	2 040	1 400	1 138
Power of operating motor, in horse-power.	70	100	70	70
Rates of speed in rising, in feet per minute	65	75	30	70
Time for full opening, in minutes	2	2	2	2
Number of cells in battery	250	250	110	250
Type of cells	F 9	F 13	F 13	F 9
Voltage of battery	520	520	230	520
Capacity of battery	40 amperes for 8 hours; final voltage, 437; 160 amperes for 1 hour; final voltage, 405	60 amperes for 8 hours; final voltage, 437; 240 amperes for 1 hour; final voltage, 405.	60 amperes for 8 hours; final voltage, 192; 240 amperes for 1 hour; final voltage, 178.	40 amperes for 8 hours; final voltage, 437; 160 amperes for 1 hour; final voltage, 405.

Warning Signals to Highway Traffic.—For signaling to highway traffic that the bridge is being raised there is a warning bell and a pair of flashing red lights on the oncoming side of each approach. These are put in action before the opening is begun, by turning a snap switch in the operating house. The switch must be on before power is available for operating the motor.

Storage Batteries.—The storage batteries are housed in well-ventilated brick buildings near the north end of each bridge. On all bridges except that at Reedy Point an ampere-hour meter in the battery leads registers the approximate quantity of stored power at all times. The indicator hand moves in one direction as power is withdrawn and returns as it is replaced by charging. An automatic trip on this meter shuts the generator set down as soon as the battery reaches the fully charged state.

Details of Construction, St. Georges Bridge.—The plans for this bridge required a symmetrical lay-out of the two piers and two abutments, as shown in Fig. 5. The contractor worked the north side of the canal with a 10-ton "whirler" derrick equipped with a 75-ft. boom and on the south side with a steel stiff-leg derrick. The caissons were 15 ft. by 41 ft. in plan. The north one was to be driven to Elevation — 43.0 and the south one to Elevation — 41.0; in each case the top of the base was at Elevation 0.0, at which point the neat work begins. An attached coffer-dam was required because this bridge crossed the canal in the upper level where the water was held at Elevation + 17.5. The cutting edge of the north caisson was set at Elevation + 9.0. The first concrete poured was allowed to set six days before the forms were stripped and the sinking was begun.

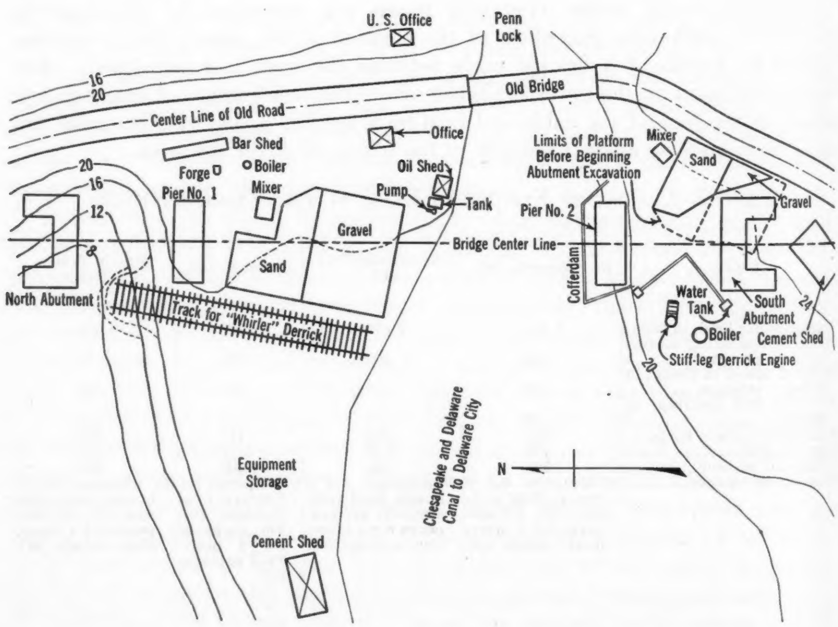

FIG. 5.—PLANT LAYOUT OF SUBSTRUCTURE, ST. GEORGES BRIDGE.

Sinking was halted when the top of the concrete was about $2\frac{1}{2}$ ft. above the ground surface, forms were set, and another lift of concrete was poured. These lifts averaged about 6 ft. in height. Forms were removed in not less than 48

hours and sinking was resumed. No unusual sinking difficulties were encountered in either of these caissons except that in the beginning the contractor's organization was indifferent to the need of checking the location adequately and directing the mucking properly. The inspection force was determined to make the pioneer job a standard for the following ones, however, and the contractor was required to plumb the north caisson which was leaning 12½ in. north and 11½ in. west, with only 4 ft. to sink before the landing elevation was reached. The cutting-edge at this point was 48 ft. below the ground surface and the material had been quite firm for the 27 ft. of sinking. The center of gravity was well below the ground line and mucking under the high edge combined with blocking under the low edge was ineffectual to right the caisson, even when two water jets were worked along the high side. All this time the caisson was sinking and was soon below the intended landing point and in acceptable material for sealing. Finally, a bridle was rigged on needle-beams and twice daily a strain was taken and clamped off, and the jetting was continued. By this method in 13 days the caisson was brought to within 2 in. of plumb in this direction. In the transverse direction the high end was loaded with earth by covering the east well and sheathing the east row of struts, thus forming a pocket inside the coffer-dam. Two jets were worked around the high end and the leaning was corrected from 12 to 1½ in. in 11 days. When accepted for plumb, this caisson was 3 in. south and 5 in. east of the center line with a 1-in. skew. The center lines of the bridge were shifted to make this pier exactly on line, and the other units were built accordingly.

The south caisson was handled in a more workmanlike manner and was landed at the intended elevation 2 in. south and 3 in. west of the line with a 1-in. skew, and 1½ and 2½ in. out of plumb to the north and west, respectively. The only unusual feature of its handling was the building of a wooden sheet-pile coffer-dam which was filled with sand in order to be able to build a dry land caisson. A pump boat was used to feed four 2-in. jet pipes during sinking.

Both caissons were sealed "in the dry" without difficulty. The working chamber was first filled with concrete to within a foot of the ceiling and allowed to stand over night. The next day the working chamber was completely filled and the wells were partly filled. A two-line submarine bucket was used for this phase of the work. The concrete was dumped at the bottom and placed by hand for the seal as long as the men could work below. On the second day, when the ceiling was reached, several batches of grout were placed and then concrete was dropped from the top of the wells, in order to churn the material close up against the roof of the chamber. The greatest distance from the side of a well to a ceiling corner was 2 ft. 3 in.

Piers.—The neat work on the piers was of conventional dumb-bell outline and presented no unusual problems. The shafts, recessed for the web walls, were poured first, and after the concrete was rubbed, the coffer-dam was braced against the shafts to permit the removal of the cross-braces so that the web wall might be poured without boxing out the bracing timbers. After this work was finished, the bolts holding the sheeting of the attached coffer-dam to the caisson were backed out, the timber bracing was removed just ahead of the back-filling, and the sheeting pulled.

Abutments.—The abutments are supported on concrete piles 14 in. square and varying in length from 16 to 26 ft. The lengths necessary were determined by driving a timber test pile of the same cross-section (14 in. by 14 in.) as the concrete piles, and 30 ft. long; these test piles were driven when the abutment excavation was at such a depth that the little additional material penetrated might be disregarded. Seven of these piles were driven for the north abutment and a contour sketch was made of the elevations of the tips when the requisite bearing of 20 tons per pile was reached. On this basis lengths were determined for pre-casting the piles. The first piles driven stopped at the depths anticipated but after the soil became more compacted, those driven later met with greater resistance. It was impossible to drive test piles even to the minimum penetration required in the coffer-dam at the south abutment. A 1½-in. jet was rigged up and five holes jetted over the area disclosed the presence of a thick layer of fine tight blue sand through which the piles would have to be jetted. Consequently, 20 ft. 3-in. piles were cast with a 2-in. pipe in the center. In driving, a hole was first made with a free jet and the pile placed therein with its inside jet flowing. At times, the outside jet was required during driving. In all cases the water was shut off when the pile was within a foot of the required depth and the remaining penetration was secured by the hammer alone, in order to have a figure for computing its supporting capacity.

The coffer-dams for these abutments were of conventional design. Tongue-and-groove sheeting was used on one and a double thickness of 2-in. by 10-in. and 2-in. by 12-in. oak sheeting for the other. The bottom of the main wall footing was at Elevation + 015, 17 ft. below the water level in the old canal, and, therefore, constant pumping was required.

The piles were cast during the winter months and were cured under steam at about 120° Fahr. They withstood driving very well. Two piles of 1 : 1.5 : 3 concrete, cured for 19 days under steam, were driven on the twentieth day with only a small amount of spalling on the head of one. No. 2 Vulcan hammer was used for all the driving; swinging leads with three guys were used and the bottom of the pile was accurately set for locating it in position.

The neat work of the abutments was of conventional design. By some overhead bracing and tying back it was possible to remove the struts of the coffer-dam timbering and pour the walls without boxing out the bracing. Material for the approach embankment was partly caisson spoil, partly clay dug by an orange-peel bucket from the canal bottom, and partly material from borrow-pits at the bridge site, but mostly it was material borrowed from pits near-by.

Erection of Steel Work.—The two towers and the lift span of St. Georges Bridge contain about 250 tons of metal each. With the floor, the lift span weighs 700 tons. Due to the necessity of not interfering with canal traffic and the fact that five panels of the lift span were over dry land prior to the final dredging operations, it was necessary to erect the span on falsework high enough to provide 55 ft. of vertical clearance over the water. All steel for this contract was transported by water and unloaded at the site by a stiff-leg derrick on the north side, which was mounted to travel on rails parallel to

the bridge (see Fig. 6). This derrick had an 85-ft. boom. Steel for the south tower was then carried across the canal on a scow to within reach of the derrick.

The bottom section of the north tower was erected by means of the stiff-leg derrick which then was used to erect the falsework for the north five panels of the lift span. A steel guy derrick with a 75-ft. mast and a 70-ft. boom, supported on falsework 117 ft. high, was then rigged to complete the erection of the north tower (see Fig. 6) and, after being moved to a lower

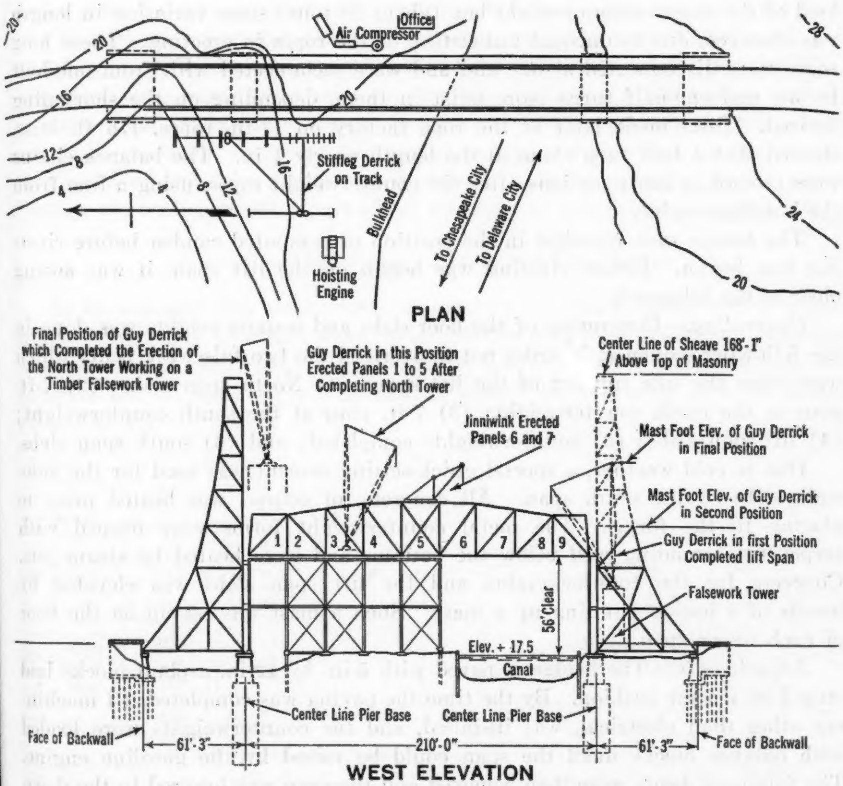

FIG. 6.—JOB LAYOUT, ST. GEORGES BRIDGE SUPERSTRUCTURE.

position, was used to erect the north five panels of the lift span. This derrick was then moved to the south side of the canal for setting the steel in the bottom section of the south tower. Its next position was on falsework inside the tower, where the bottom of the mast was about 36 ft. above the top of the masonry. From this point a falsework bent was built to carry the south end of the lift span. During this time a "jinniwink" working from the top chord of the lift span, and fed by the stiff-leg derrick, placed the steel of the two panels south of the center panel, cantilevering over the channel. The guy derrick then completed the lift span and south tower in two moves.

Steel-plate counterweight forms were assembled on the floor of the approach spans with the reinforcing and counterweight girders inside. Two counterweight ropes were passed over each sheave, by a gin-pole mounted on top the tower, and connected through the equalizers and eye-bars to the counterweight girder. The opposite ends of these ropes were then connected to two sets of blocks reeved for two lines to a hoisting engine, and the counterweight box with the girder was raised until the ropes could be attached to the lifting girder. The gin-pole then placed the remaining ropes over the sheaves. After all the counterweight ropes had been connected and allowed to carry the full load of the empty counterweight box (about 20 tons) some variation in length was observed, due to unequal untwisting of the ropes in erecting. These long ropes were disconnected at one end and were reconnected with from one-half to one and one-half turns more twist in them, depending on the shortening desired. Tests made later at the rope factory on $1\frac{3}{4}$-in. ropes, 176 ft. long, showed that a half turn changed the length nearly $\frac{1}{2}$ in. The balance chains were erected in short sections after the counterweight ropes, using a line from the hoisting engine.

The towers were plumbed in the position of computed camber before riveting was begun. Before riveting was begun on the lift span, it was swung clear of the falsework.

Concreting.—Concreting of the floor-slabs and counterweights was done in the following sequence, in order not to overload the two falsework bents which were then the sole support of the lift span: (1) North span slabs; (2) 7-ft. pour at the north counterweight; (3) 7-ft. pour at the south counterweight; (4) lift-span slabs; (5) counterweights completed; and (6) south span slabs.

Due to cold weather, a special quick-setting cement was used for the sidewalk slabs of the south span. All concrete, of course, was heated prior to placing in the forms. The metal counterweight forms were draped with tarpaulins extending well below the bottoms and were heated by steam jets. Concrete for the counterweights and the lift span slabs was elevated by means of a bucket running up a mast. Such a mast was set up on the floor of each tower span.

Adjustments.—The bridge is paved with 5-in. by 12-in. asphalt blocks laid on a $\frac{1}{2}$-in. mortar cushion. By the time the paving was completed, all machinery other than electrical, was installed, and the counterweights were loaded with balance blocks until the span could be raised by the gasoline engine. The falsework bents were then removed and the span was lowered to the down position; the shoes were grouted to proper elevation; the expansion plates on the fixed spans were set to match those on the lift span; and the span and the counterweight guide casting were set and attached. One of the castings had been attached prior to the pouring of any counterweight concrete. After beginning operation, it developed that that method was improper because the counterweight did not hang exactly as had been expected and the guide casting wore quite rapidly. It had to be replaced by one properly set.

Traffic was turned on the bridge on March 1, 1926, to permit the removal of an old lock and the bridge was operated by the auxiliary power unit pending the completion of electrical installation. Since the movement was only 10 ft.

of travel per min., much adverse comment was forthcoming from the traveling public.

The installation of electrical equipment was completed before commercial power was made available for operating the motor generator set for charging the battery. After the battery (250 cells of 9 plates each) was set up and the electrolyte poured, an attempt was made to make the initial charge by driving the generator by the gasoline motor, but to charge the entire battery as a unit was impossible, because the belt drive, from engine to generator, was too short and was in the wrong direction to transmit full load. Consequently, the battery was split in half for charging purposes and each half was charged for twelve hours each day continuously except for time required for making lifts. Ten days were required to raise the specific gravity to a point high enough to make it safe to leave the battery inactive. When commercial electric power was made available, operation was begun with the use of regular equipment. The speed of travel in lifting was increased to 60 ft. per min., with a satisfactory shortening of delay to traffic. The cost of this bridge was $354 000.

Substructure of the Chesapeake City Bridge

At Chesapeake City the bridge crossing was over Back Creek, which is tide water. Both pier locations were in the stream and both abutment locations were covered at high tide. The contractor elected to work each side with his favorite bit of equipment, a 10-ton "whirler" derrick with a 75-ft. boom. (See Fig. 7.)

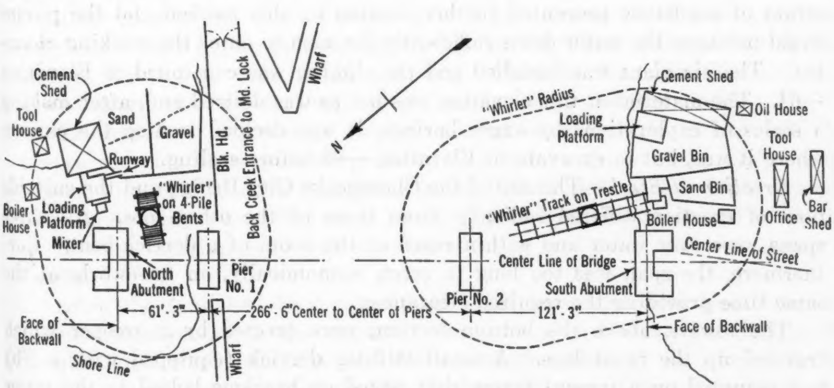

Fig. 7.—Plant and Job Layout, Chesapeake City Bridge Substructure.

Caissons.—The pier bases were 16 by 43 ft. in plan and were built 42 ft. and 49 ft. high for the north and south piers, respectively. The planned elevation of the tops was —15 (mean low water at Chesapeake City is Elevation +2.3 and mean high water is Elevation +4.5). Floating caissons 26 ft. high were built on land ways and launched at high tide. A composite steel and timber construction was used; the cutting-edge, walls, and roof of the working chamber and wells for a height of 20 ft. were of steel-plate and angle construction as was some bracing in the bottom 6 ft. The outside faces

above the cutting-edge were of 3 by 12-in. tongue-and-groove oak sheeting supported by 12 by 12-in. timber bracing.

Both caissons were adequately caulked and no difficulty was encountered in towing them to position. The work of sinking the floating caisson to the creek bottom was begun on an ebbing tide until 6 ft. of concrete had been poured. At first, difficulty was experienced in keeping the caissons plumb because the 1-yd. concrete bucket was too large for the narrow caisson openings. Open-well dredging was then begun and concrete was poured as required. Attached coffer-dams, similar in detail to those used at St. Georges, were used above the tops of the caissons.

The south caisson was sunk by open-well dredging until it became apparent that proper control was impossible because of buried logs. The equipment for pneumatic work was then installed. There were three wells, 7 ft. in diameter, and one man-lock, one muck-lock, and one concrete lock. This caisson was sunk under air from Elevation —28 to Elevation —42, when it was necessary to shift the pneumatic equipment to the north caisson. Open dredging was again attempted and abandoned as unsatisfactory. After the north caisson was sealed, the air equipment was returned to the south caisson, which was uneventfully driven to Elevation —57 and sealed in a hard blue clay. For the sealing, both the concrete and muck-locks were used; the working chamber was filled to within 1 ft. of the ceiling and allowed to set under pressure for 60 hours before removing the locks and concreting the wells.

The north caisson was sunk to Elevation —42 by open dredging, where strata of sandstone prevented further sinking by this method and the pumps could not keep the water down sufficiently for men to enter the working chamber. The air plant was installed and the sinking was continued to Elevation —64. The material at this elevation was not as was desired and, after making a series of explorations by auger borings, it was decided to stop the caisson where it was, but to excavate to Elevation —69 before sealing.

Erection of Steel.—The site of the Chesapeake City Bridge and the suitable plan of erection differed radically from those of the other jobs. All three spans were over water and within reach of the boom of a derrick-boat. Furthermore, the span was too long to erect, economically, on falsework, at the same time providing the required clearances.

The towers, above the bottom section, were erected by a creeper which traveled up the front legs. A small stiff-leg derrick (equipped with a jib) was mounted on a trussed frame that rested on brackets bolted to the tower columns. The entire unit was hoisted by cables reeved through blocks attached to the tops of the last erected column sections. The sheaves were set and the ropes hung by this creeper.

The counterweight forms, of the same metal arrangement as those at St. Georges, were assembled resting on a grillage supported by temporary brackets attached to the tower front and rear legs. In this position the counterweight was about 1½ ft. below its position with the span seated. After the towers were riveted the counterweight concrete was poured sufficiently high in the forms so as not quite to balance the metal and machinery of the lift span.

The balance chains could not be attached until after the supporting grillage had been removed.

The lift span was erected on three scows anchored in front of the south pier and was protected by a system of pile clusters. The operating machinery was installed and the down-haul ropes run on the drums. The up-haul ropes had been attached to the tower tops. The falsework on the scows had been detailed so that, at high tide, the span would be high enough to permit attaching the ropes, already fastened to the counterweights, to the lifting girder and, by partly flooding them at low tide, the scows would lower the span to its position on the shoes and be low enough to pull out from under.

On January 23, 1926, canal traffic at this point was suspended and the lift span was floated into a position where it should have been possible to move it sidewise into place over its shoes. An interference developed which required the removal of four deflector sheaves and a steel member in the roadway floor system at the north end before the span was coaxed into position. The counterweight ropes (sixty-four in all) were connected and after the scows were taken from under the span, the supports upon which the counterweights had been built were removed, and the installation of operating ropes was completed. Thereupon, an effort was made to raise the span by means of the 40-h.p. gasoline engine. The first effort was unsuccessful, due to extra weight that had been placed on the span to insure its picking the counterweights off their supports. After the removal of some of this weight, the span was picked on the second attempt with the engine fully loaded. As the span was raised and the weight of the counterweight ropes was transferred from the span to the counterweight side of the sheaves the engine found the load easier and easier to pull until after having lifted the span 86 ft. the balance was nearly even. The span was lashed at this point and was not moved again until all the concrete and paving work was completed.

The approach spans were riveted as soon as they were swung and the towers as soon as they were plumbed and before the counterweights were poured. On the lift span the lifting posts, lifting girders, floor-beam connections, and stringer connections at alternate floor-beams were riveted while the scows were carrying the load; all other riveting was done after the span was swung.

After the span had been raised and canal navigation had been resumed, an investigation as to the cause of the lack of end clearance during the "floating in" was made. It was discovered that through an error in the drafting-room, four members in the eleven-panel trusses, had been fabricated 2⅝ in. too long. To replace them was impracticable and it was necessary to shift all the deflector sheaves 2⅝ in. and to reinforce all top chord splices (many of which were partly open) so that the stresses might be computed as being carried entirely by rivets.

On April 24, 1926, all metal work, concrete, paving, and machinery installation had been completed, and the bridge was opened to traffic to allow the removal of the old bridge. On the next day, while operating in a heavy wind, a balance chain fouled on a horizontal brace of the north tower, and broke. The falling chain and the tower were severely damaged. The bridge was

closed to traffic and replacement parts and material for a revision of details of the chains and towers were ordered. The approaches to this bridge were constructed by the State of Maryland. The cost of the bridge was $476 000.

Substructure of Summit Bridge

Summit Bridge is located 5 miles east of Chesapeake City, in what is known as the "Deep Cut". The banks at this point are 80 ft. high and it was necessary to excavate 56 000 yd. of dry material so that the substructure could be put in and the approaches made. This was an extra contract item and was performed by steam shovel and trucks in a period of four months, before actual work on the regular contract could begin. The bridge rests on four piers as shown in Fig. 8. Piers Nos. 1 and 4 are spill-through abutments, and Piers Nos. 2 and 3 are essentially of the dumb-bell type, resting on solid concrete bases.

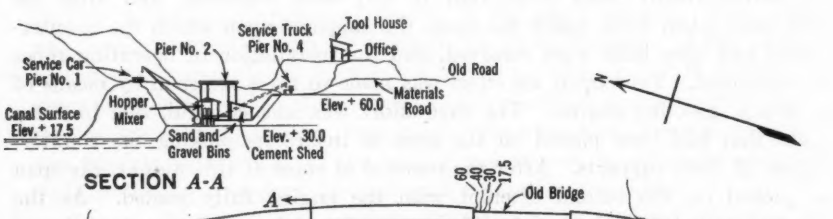

FIG. 8.—PLANT AND JOB LAYOUT, SUMMIT BRIDGE SUBSTRUCTURE.

The contractor used two 10-ton "whirlers" with 75-ft. booms for Piers Nos. 2 and 3, and two stiff-leg derricks for Piers Nos. 1 and 4. A central mixing plant was used for concreting. The two "whirlers" handled materials and placed the concrete for Piers Nos. 2 and 3, but trucking was necessary to convey the concrete to Piers Nos. 1 and 4.

Caissons.—The pier bases were built by sinking concrete caissons, 20 by 46 ft. in plan, with three 7-ft. wells. These wells are of the same type as those described for the bridge at St. Georges. The base for the south pier was planned for a height of 49 ft., but was built 55 ft. high. The base for the north pier was planned and built with a height of 43 ft. The attached coffer-dams were similar in detail to those at St. Georges Bridge.

The shoe for Pier No. 2 was set at Elevation +41 and the first pour made. After the usual 6-day curing time, the work of sinking the caisson was begun. At Elevation —27 it became necessary to extend the attached coffer-dam (which was already 24 ft. high) by an additional 20-ft. section. The sinking of the caisson was resumed, but friction on the sides of the attached coffer-dams became so great that three days of jetting and jarring with dynamite were necessary to start the caisson moving. Steel drums 7 ft. in diameter were placed around the wells, and the caisson was loaded with mud for added weight to aid sinking. At Elevation —37 the west end of the attached coffer-dam was seen to be pulling free from the caisson and sinking was stopped. At this point the top of the pier base was at Elevation +6 and the ground surface at Elevation +41, which gave a vertical height of 35 ft. of attached coffer-dam subject to skin friction. Thirty days were lost while 45-ft. steel sheet-piling was secured and driven around the caisson. This sheeting was driven quite close to the attached coffer-dam and penetrated to Elevation —4, which was 4 ft. below the final elevation of the top of the pier base. Sinking was resumed inside the steel-pile protection, and the caisson landed at Elevation —42.8, $\frac{7}{8}$ in. east of line and $1\frac{1}{2}$ in. north, with $3\frac{7}{8}$-in. skew. Borings were made to investigate the bottom material and it was found to be satisfactory. The caisson was sealed "in the dry," the material being placed along the cutting-edge by hand and, finally, dropped from the top of the wells to insure the proper filling of the working chamber.

Abutments.—Piers Nos. 1 and 4 were excavated to a depth of 40 ft. by hand. The material was loaded into buckets which were handled by the stiff-leg derrick. An open timber coffer-dam was used and at Elevation +40 considerable water was encountered. The foundation was excavated a foot lower than grade and a mat of dry concrete was placed to assist in controlling the water and to provide proper material on which to pour the reinforced footing. A sump for pumping was left, and when the footing had been placed the sump was plugged. The shafts were completed next. Owing to the appreciable quantity of ground-water present, it was thought advisable to provide some means of handling it, should it tend to uncover the footing. To this end a wall 4 ft. high was poured between the piers and beyond to the end of the footing, with drain pipes placed so that a connection might be made to them. Thus, the water might be carried away without the possibility of erosion close to the pier.

Erection of Steel Work.—The erection plan called for the use of falsework bents under the entire structure, the lift span being erected in the down position. This plan left a clear height of 56 ft. through the then existing canal channel which passed under the south approach span. One bent of falsework was omitted under the fifth panel, leaving a clear width of 25 ft. between bents. Timber fenders were built to protect the pile bents in the narrow channel.

The steel and machinery were delivered by rail at Mount Pleasant, Del., two miles from the bridge site, by a concrete road, and were trucked to the job where the traveler unloaded and sorted them in the yard.

A 10-ton traveler with 50-ft. boom was used to set the falsework as well as the steel in the floor system and the trusses of the three spans. When the spans were completed a guy derrick was set on falsework on top of the north end of the lift span and by the use of a 100-ft. boom all the north tower steel was placed. The derrick was shifted from the north to the south end of the lift span in 8 days, and the south tower was erected in 14 days thereafter. The cost of this bridge was $475 000.

Substructure of Reedy Point Bridge

The Reedy Point Bridge was built over the already dredged, but not then used, new entrance south of Delaware City. The abutment and pier details were quite similar to those at St. Georges, except that timber piles were used under the abutments instead of concrete. (See Fig. 9.) The site was level and entirely free from obstruction of any nature. Abutment excavation was in a light sand and jetting of the piles was not necessary; 26-ft. piles reached the required bearing value of 20 tons.

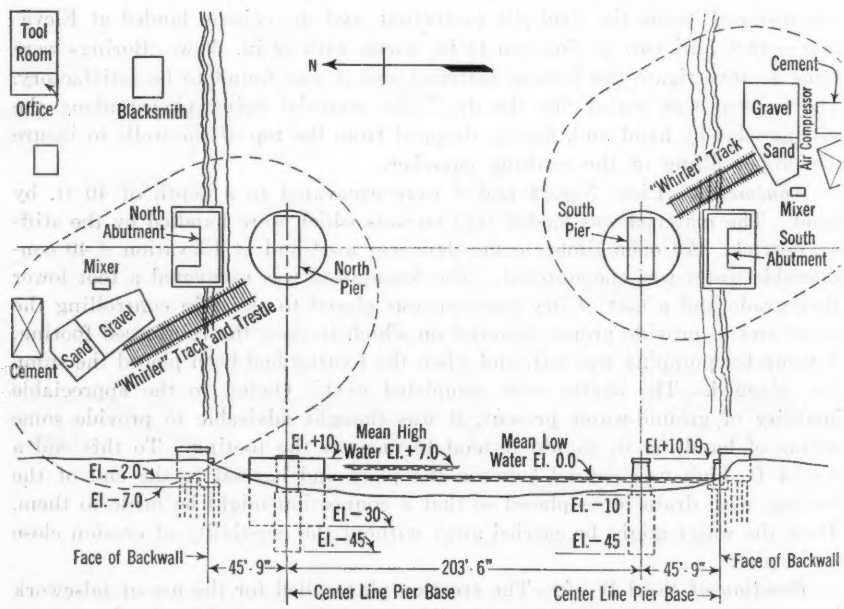

Fig. 9.—Plant Layout, Reedy Point Substructure.

Caissons.—In order to avoid the building of floating caissons, circular coffer-dams, 43 ft. in diameter, were driven in 40-ft. lengths of $13\frac{1}{2}$-in., straight web steel sheet-piling to Elevation —30 and filled up to above high water with sand dredged from the canal. The caissons were built on these islands. Excavation was done partly by orange-peel bucket and partly by hand.

Erection of Steel Work.—The shore lines, at high tide, were against the abutment faces; the channel was closed to all traffic and the Government loaned the contractor a 10-ton derrick-boat; these last named features per-

mitted simple falsework erection (3-pile bent) and the building of the lift span complete, including concreting and paving, in the down position.

Some steel was delivered by barge from the rail-head at Delaware City, but mostly it was trucked from Wilmington, Del., and unloaded by the traveler brought from Summit for the purpose.

The bottom sections of the towers were erected by the derrick-boat on the south side and by the traveler on the north side. The same creeper that was used at Chesapeake City, revised for the narrower width, was used to erect the remaining tower steel and no difficulty was encountered. The sheaves and the counterweight ropes were set by means of the creeper before it was finally lowered to the ground. The lift span was erected in part by the derrick-boat and in part by the traveler. The cost of this bridge was $318 000.

Bridge of the Pennsylvania Railroad Company

By agreement with the Chief of Engineers, United States Army, the Pennsylvania Railroad Company arranged to rebuild its bridge across the canal. The new railroad bridge was to be of the same general type and clearances as those adopted for the highways, and pier foundations were to be carried to the same depth. Detailed studies of plans were initiated at an early date by the engineers of the Railway Company. A slight change in the location of the bridge was found to offer decided economy and greater facility for caring for both canal and railway traffic during construction. The change involved shifting the center line of the bridge about 250 ft. southward from the old draw-span so that the new bridge was constructed end to end with the old one. The construction was completed before the excavation of the new canal bed below it. The new canal alignment was changed to conform to the new layout.

Under the agreement the entire work of reconstruction was to be undertaken by the Railway Company subject to the approval of the District Engineer; the Railway Company was to be reimbursed by the United States to the extent of two-thirds of the cost.

The plans for this bridge provided for a double-track structure of the vertical lift type. The main piers were 211 ft. apart from center to center; the north tower, or approach span, 172 ft. from center of rest pier to center of main pier; and the south tower or approach span, 106 ft. from face of abutment to center of main pier. The site of this bridge was occupied by an approach embankment leading to the old bridge, making it possible to construct the new structure around the old track.

To carry the tracks over the pits in which the main piers were sunk, temporary girders, supported on pile abutments, were installed, and switches were provided to shunt all regular railway traffic to the northbound track on the bridge. The southbound track was left free for use of the necessary construction plant engaged on the new work.

In all essential elements, the design of this bridge was the same as that of the highway bridges, except for the heavier loading. The main piers were sunk by the same process and to the same depths as those for the highway bridges. The foundation material was not as satisfactory as could have been desired (especially that in the north main caisson) and, later, gave rise to

troublesome pier movements. There was, however, no promise of finding better material by going deeper, so it was necessary to make the best of the material encountered. The share of the cost of this bridge paid by the United States was $552 000.

Pier Movements

The St. Georges, Summit, and the Pennsylvania Railroad Bridges were constructed prior to the excavation of the new canal prism past their sites, and subsequent to or during such excavation, movements of the main piers occurred in all three cases. The principal movement in each case occurred on the north pier, and in the latter two cases it was sufficient to interfere with the operation of the bridge. The maximum movement seems to have been about 3 in. horizontally, that is, the piers moved toward the channel that much, while the railroad bridge pier not only moved horizontally about the same amount, but also settled about 3 in. This movement in each case appears to have been due to a slight tipping of the pier due to a re-adjustment of earth pressures after the foundation material had been disturbed by the excavation of such a large prism. It was necessary to resort to corrective measures in each case in order to render the bridge operative without danger to either canal or land traffic.

The railway bridge was raised by jacking, and the north tower was moved back enough to give proper clearances. Shims were placed under it to raise it to proper grade. After this operation it worked satisfactorily.

A different method was used on the St. Georges Bridge, where a horizontal movement alone had occurred. A pump-boat capable of supplying water for seven or eight jets of 1½-in. pipe was available, and jets were sunk on the land side of the piers until the material was loosened all the way down. At the same time a 3-in. steel cable was passed around the pier in a loop, and the two ends were brought back to substantial "dead men" located near the abutments. These ends were attached to screw-jacks working on the "dead men" to give a strain on the cables. The cables were tightened as the jetting proceeded, and, in this manner, the pier was slowly brought back into its proper position. The same method, with suitable variations, was then successfully applied to the Summit Bridge.

Economic Value of Canal

The soundness of a prediction for an early demand for an increase in the capacity of this waterway has been questioned. Antagonists of waterway improvements have declared it a waste of public funds. This is not the case in the instance of the Chesapeake and Delaware Canal. Assuming that a public work like this is for public good, for public convenience, and for public economy, this canal makes an excellent showing for the funds expended on it. Omitting all consideration of pleasure boats, passengers, and empty vessels, it gives passage to from 600 000 to 700 000 tons of freight annually. Knowing the classification of this freight, its points of origin and destination, and the proper carriers rates, it is possible to compute the actual cost of transporting that freight by water, and to compute what it would have cost if shipped by

rail, thus giving the direct saving, if any, due to the existence of the canal. This computation was made in 1926 for the canal for the entire period since it was taken over by the United States in 1919. From the annual savings thus computed there was deducted all actual operating costs of the canal, an assumed sinking fund of 2% to amortize the total cost in fifty years, and interest at 4% on all appropriations. After making these deductions, the net savings in freight transportation costs (which is thus added to the National wealth) is in round numbers $1 000 000 per year, or, say, $1.25 for every ton of freight passing through the canal.

Hydraulics

Prior to the completion of the canal, considerable interest from technical and practical standpoints attached to the question of the tides and currents that might be expected after the completion of the projects. The question involved a study of the result of the interference of two unequal opposed waves in a canal of limited length and of practically constant cross-section. There is little knowledge in the engineering world of the resultant phenomena in such cases. Navigators are interested in them from a practical standpoint, since they will have a bearing on the passage of vessels through the canal. Early attention, therefore, was given to studies of phenomena to be expected with a view to predicting them beforehand, and subsequently checking the predicted results by observing the actual phenomena.

The canal was to be 90 ft. wide on the bottom, with side slopes of 1 on 2. It was to be somewhat wider at the two ends, but this extra width was neglected in studying the problem. The depth was to be 12 ft. at mean low water, referred to the Sandy Hook datum plane; that is, its bottom was to be level. It has only a few curves, which are of long radius, and these were also neglected. Its length from Delaware River to Back Creek is 13.5 miles, and it extends down Back Creek (a slowly widening stream) for 4.5 miles. Back Creek was omitted from consideration in the computations for the sake of simplicity. This omission undoubtedly involved some error, but how much is not known. It is thought to be small. The major tide entering the canal (an average amplitude of 6 ft.) comes from the Delaware River. The minor tide entering from Chesapeake Bay has an average amplitude of 2.4 ft. at Chesapeake City. Mean tide level is at substantially the same elevation at each end. The smaller tide reaches Chesapeake City two hours earlier than the major tide passes the jetties in Delaware River. The bottom of the canal is 15 ft. below mean sea level.

There are many localities where unequal and opposed tides meet in natural channels, but in most such cases the phenomena peculiar to this problem are so masked by others due to purely local conditions (such as varying widths and depths) that very little progress has been made in their study, and but few studies are available in the English language.

Canals of fairly constant depth and cross-sectional area are usually artificial; and it is to this class that engineers must look principally for a solution of the problem proposed.

The Cape Cod Canal, which is an artificial cut connecting Cape Cod Bay with Buzzards Bay, in Massachusetts, presents the problem in a manner very similar to the Chesapeake and Delaware Canal. It has been described by William Barclay Parsons,* M. Am. Soc. C. E., who based his study on the writings of Airy, de St. Venant, and Maurice Levy. The equations he gives are substantially those of Levy.

Probably the best study of the problem in a natural channel is that of the English Channel by M. Bourdelles,† who does not attempt to solve the problem mathematically; but his explanation of the theory is excellent. By following the theory of M. Levy as given by Mr. Parsons, the writer has computed the tides and currents to be expected in the Chesapeake and Delaware Canal. The predicted tides as given by this computation are shown graphically in Fig. 10 (*a*), and the predicted currents are shown in Fig. 10 (*b*).

A study of the assumptions made in the deduction of the Levy equations, however, led the writer to doubt their validity, especially in the case of such a shallow canal as the one under consideration. He concluded that: (1) The wave propagated from each end should be considered merely as a wave of translation, with a positive phase during high water and a negative phase during low water; (2) its height at any point in the canal should vary sinusoidally as did the generating waves in the respective bays; (3) each of these waves of translation contained a certain quantity of energy; and (4) one-half the energy was potential and was represented by the height of the wave above mean sea level, and the other half was kinetic, represented by the velocity of the water at a given point; (5) this energy was expended in overcoming friction in accordance with a law which was deduced, resulting in a consequent falling off of height of wave and of velocity as the wave was propagated farther into the canal; (6) the resultant height at any point in the canal at any instant was the algebraic sum of the instantaneous heights due to each wave at that point; and, (7) the resultant velocity at any point was the algebraic difference between the instantaneous velocities due to each wave at that point. Fig. 11 shows the resultant heights and velocities as computed under these assumptions.

There are no striking differences in the results. Both indicate that (*a*) tidal propagation is not instantaneous; (*b*) in general, the canal surface between the two ends will be concave on a rising tide and convex on a falling tide; (*c*) the larger tidal oscillation at the Delaware River progressively decreases in amplitude to a point about 2½ miles from Chesapeake City; (*d*) the resultant wave is propagated in the direction of west to east, or against the direction of propagation of the major component wave; and (*e*) the speed, or rate of travel of the wave, is very small at the western end but extremely rapid at the eastern end.

Some discepancies are found between the two results. The Levy formula gives symmetrical curves for high and low water, while the wave-of-translation formula does not. It gives amplitudes of greater magnitude for the positive

* "The Cape Cod Canal," by William Barclay Parsons, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), p. 1.

† "Etude sur le Régime de la Marée dans in Manche," Bourdelles, *Annales des Ponts et Chaussees*, 1899, 3me Trimestre.

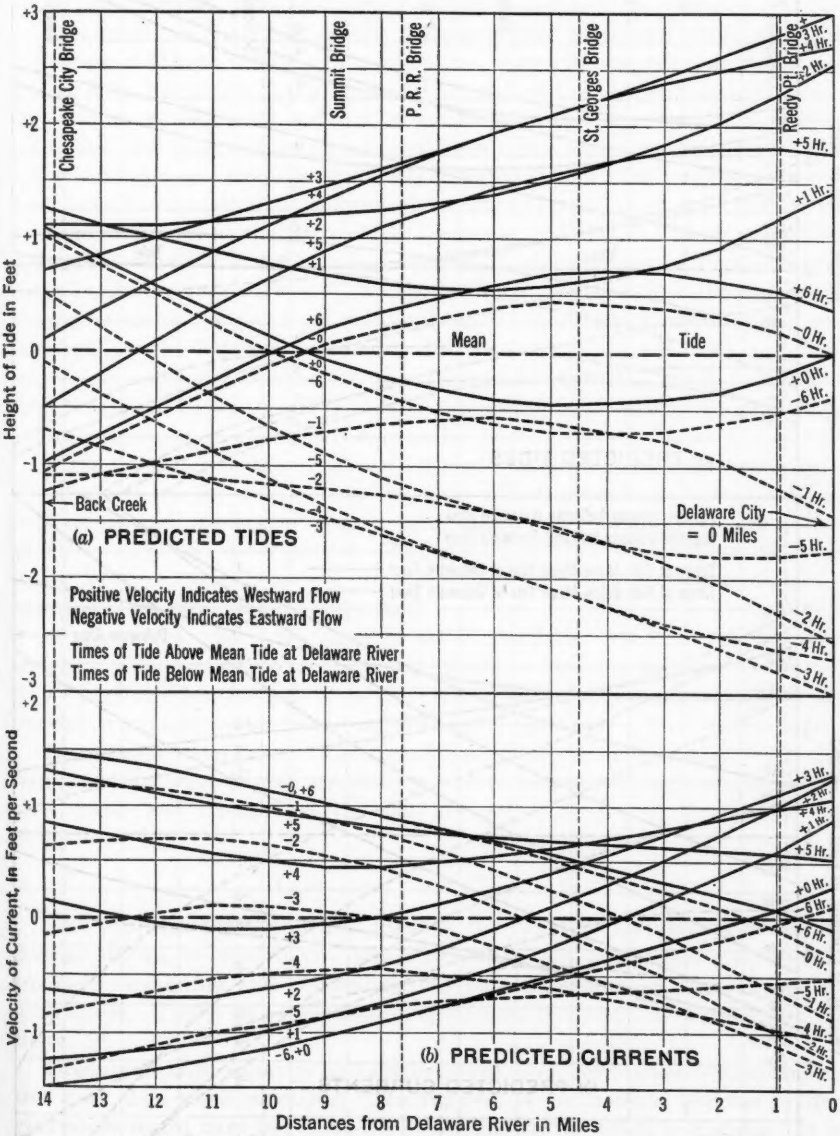

FIG. 10.—COMPUTED INSTANTANEOUS VELOCITIES OF TIDAL FLOW FOR EACH HOUR OF TIDE (BY PARSONS' METHOD).

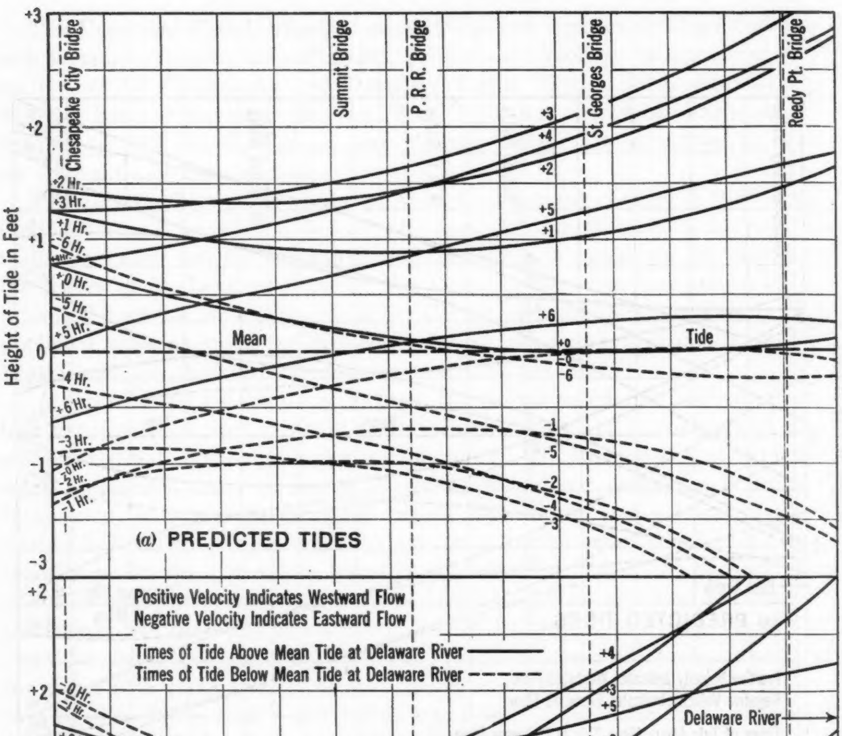

FIG. 11.—INSTANTANEOUS VELOCITIES OF TIDAL FLOW COMPUTED BY THE WAVE-OF-TRANSLATION FORMULA.

phase than for the negative. A comparison of the two sets of predicted heights at any given point shows but little difference in amplitude for the positive phase, the principal variation being manifest in the negative phase.

Greater discrepancies are shown in the predicted velocities. Both agree in indicating that zero velocity, or slack water, comes at varying heights of the tide at different points in the canal. It is predicted to occur at or near mean sea level at the Delaware end and at or near high or low water at the Chesapeake end, and that neither the time of slack water nor of the strength of the tide is simultaneous throughout the canal, but that they both progress at varying rates. Both predictions indicate times when the current will be flowing in opposite directions at the two ends of the canal, and that it will be nearest to having a uniform velocity in all parts of the canal at times of high and low water in the Delaware River. The most striking discrepancy is the difference in magnitude of the predicted resultant velocities. Quite high velocities are indicated for the Delaware end in Fig. 11 (*b*), decreasing to Mile 10, and then increasing again at Chesapeake City. The greater velocities are indicated for the Chesapeake City end in Fig. 10 (*b*), decreasing at a small figure at Mile 5, and increasing again at the Delaware end.

After the water in the canal was lowered to sea level, no opportunity presented itself to the writer to make a complete comparison between these predictions and the actual phenomena, because the slides in the canal frequently varied the cross-section. The results of such observations as were made, however, were deemed quite favorable for indicating what might be expected in an ideal state of the canal.

A comparison shows that neither set of predicted results agree with the observed phenomena as closely as they should. For instance, the geometric loci of high and low water have no similarity, and the maximum velocity of the current at various points in the canal is fairly constant.

Observation does agree with prediction, however, in showing that the resultant tidal wave is propagated from west to east at varying rates, and that slack water and strength of the tide are not simultaneous throughout the channel.

Conclusions and Comments

The requirements that traffic through the canal be maintained as far as possible during re-construction was a necessary one, but caused considerable trouble in planning and controlling the simultaneous operations of several contractors. Traffic and construction operations had to be co-ordinated at all times.

Dredges of large capacity and ample power were much more economical on this work than small machines. The removal of all material possible by dry excavation might have had some effect in reducing the activity of slides. It is believed that in future enlargements of the canal this will be necessary.

Less expensive bridges might have been constructed as temporary expedients thus effecting a considerable reduction in present first cost of the canal. The course followed, however, was in the opinion of all concerned the one having the greater ultimate economy.

The provision of an ample right-of-way for the disposal of spoil is exceedingly desirable where land is relatively cheap. Any damage to adjacent private property leads to vexatious disputes, exhorbitant demands for compensation, and possible litigation.

Wet borings are absolutely unreliable for determining the character of the sub-soil. Core borings only should be depended on.

Tidal variations of level and currents in the canal are on the whole detrimental to navigation. The detriment lies not so much in their effect on speed and time of transit, as on safety to the vessel. If a large and heavily laden vessel grounds and gets athwart the channel, as it may readily do under action of currents, a fall in the tide may break the vessel in two. Tows approaching closed bridges with a fair wind and tide, and having to check their speed on account of failure of the bridge to open, are particularly exposed to having one or more vessels of the tow ground.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

REPORT OF THE COMMITTEE OF THE IRRIGATION DIVISION ON "A NATIONAL RECLAMATION POLICY"

Discussion*

BY MESSRS. A. D. LEWIS AND JOSEPH JACOBS.

A. D. LEWIS,† ESQ. (by letter).‡—After a tour of inspection in America in 1913 the writer commented upon the irrigation situation as follows:§

"The result achieved in settlement is, after all, the measure of success in irrigation. Failures in irrigation schemes have resulted not so much from bad engineering as from lack of a proper understanding of the agricultural conditions and of all that is involved in the human aspect of settlement". * * * "The chief lessons learnt were that settlers are not attracted to these schemes as rapidly as was expected and that difficulty is experienced in getting full payments from a large proportion of the settlers. * * * The supply of settlers with sufficient experience and capital is limited. * * * It is worthy of note that the American Government has not yet adopted the policy of loan advances (in cash or stock) to settlers to help them over the first few years."

During the sixteen years that have elapsed, the U. S. Reclamation Act has frequently been modified, numerous Commissions have reported, and the Reclamation Service has been unjustly abused and re-organized, but it appears that no complete solution of the difficulties noted in 1913 has yet been evolved in America. The history of Government efforts in irrigation has been very similar in South Africa, although conditions are less favorable to success. The surface of the country is very steep and denuded. The supply of water from annual rainfall and other sources is very small and erratic, and the cost of constructing storage works is excessive. The entire problem is further complicated by a heavy silt load in streams. The population density is less than 10% of that in the United States; marketing conditions are worse; and the tendency of political agencies to rush forward with new projects or to obtain special patronage for old ones is at least equal to that in America.

* Discussion of the report of the Committee of the Irrigation Division on A National Reclamation Policy, continued from December, 1929, *Proceedings*.

† Director of Irrig., Union of South Africa, Pretoria, Union of South Africa.

‡ Received by the Secretary, October 2, 1929.

§ "Irrigation and Settlement in America," by A. D. Lewis.

Despite these difficulties, irrigation received a tremendous impetus because of the ostrich feather industry, which is peculiar to South Africa. Dealers were prohibited from exporting the birds and this resulted in a very profitable monopoly of the market. Alfalfa was necessary for raising ostriches and it could be grown locally only under irrigation. From 1906 to 1913 the average price of feathers exceeded \$13 per lb. and profits of at least \$300 per acre yearly were made. As a result of these large profits, vineyards and other established crops were uprooted from the existing irrigated lands and were replaced by alfalfa crops. There was a frantic rush to build new irrigation works, many of which were merely primitive furrows to divert the intermittent flood flows from rivers. The industry was based on a fickle, feminine fashion and "all the eggs were in one basket." The standard of profits was far greater than the general average for irrigation farming. In 1914, fashions changed, the World War broke out, a severe drought occurred, and the industry crashed. The value of the export of feathers fell from \$15 000 000 in 1913 to less than \$20 000 in 1918, and a post-war agricultural depression supervened.

From 1914 there was a feverish demand for storage works generally to alleviate the unfavorable conditions and especially to save the farmers who had spent their capital on flood-diversion projects. To date the total expenditure for irrigation in South Africa is about \$35 000 000 and nearly three-fourths of this expenditure was authorized by Parliament during the period from 1914 to 1920. It is now almost certain that the Government will not recover even half of this sum. Had the works been spread over a longer period and had active steps been taken to check speculation and to insure the rapid settlement of the area brought under irrigation, there might have been hope of recovering a larger part of the expenditure, but certainly not all of it. Since 1924 very few new projects have been sanctioned and the question of subsidizing future works is now under consideration.

Some of the projects are built and operated by the Government, but most of them are managed by co-operative irrigation district boards to which the Government has loaned money at 5%, payable over a period of thirty or forty years. Most of the land is privately owned, and the unit costs per acre of irrigable land have been about \$130, which is probably double the cost of projects in the United States. In South Africa the physical difficulties are greater, but the higher prices that prevailed after the war (when most of this work was done) accounted for part of the higher costs. The better projects, however, were begun first and the unit cost of future developments in arid areas is likely to be even higher.

It will be seen, therefore, that difficulties in South Africa and in the United States are very similar. Engineers in South Africa have followed every phase of irrigation activities in America, always watching for the discovery of some magic process which, without any direct financial loss to the Government, will rapidly convert the desert places into smiling gardens peopled with healthy and prosperous families. Lack of success in the United States warrants the fear that there is no such process.

By waiving interest charges, irrigation projects of the U. S. Reclamation Service have been subsidized at least to the extent of 40% (the ratio is

apparently more than 70%, according to Mr. Lippincott*). In addition, considerable capital and operating expenses have been cancelled. That is a sad picture which, after twenty-seven years of effort, can only be described as direct financial failure. Some of the causes can be remedied, but others will continue for many years, and the conclusion seems inevitable that under present conditions Government effort in irrigation cannot be made profitable in a literal sense. Similar effort in the immediate future will involve heavy financial subsidies. The case for the immediate extension of irrigation on a considerable scale will have to rest on the indirect advantages, particularly those that are expected to accrue after a longer period even than a generation or two.

Throughout the world irrigation on a large scale has only been immediately successful under two essential conditions: (1) When the physical aspects are such that the works would be exceedingly cheap; and (2) when there is a large and frugal population, crying out for agricultural expansion because the opportunities for their absorption into other more profitable forms of occupation are restricted.

India fulfills both these conditions. Spain and Italy fulfill the second, and, in regard to some of the earlier works, the first condition; but in later works both countries have found it necessary to resort to subsidizing. The density of population in America is scarcely one-third that in Spain and one-sixth that of Italy; the relative density of the agricultural population is still less; the standard of living is far higher; and towns and industries continue to offer a more attractive occupation. The problem of attracting settlers to irrigated lands is only a part (the more expensive part) of the general problem of encouraging agricultural settlement and development; and as long as the Government attempts to force the natural tendencies of a population it is likely that subsidizing in some form will be necessary. Judging by results throughout the world and by the standard of direct financial success, it seems as if the United States has been premature in anticipating a genuine land hunger. South Africa has been still more premature in attempting to recover the entire expenditure, with interest, under much less favorable physical and economical conditions and an even weaker intensity of land hunger.

Without entering into the question as to whether it is desirable that irrigation should be pushed ahead by Government, even if it involves an immediate direct financial loss, it should at least be pointed out that in such countries as Spain and Italy, where irrigation is an old problem, the greatest prosperity is generally to be found grouped around the old irrigation works. The road to this prosperity has seldom been an easy one. Safety and security have often been reached only by a process resembling that by which the soldiers of Cortez crossed the watery gaps in the Mexican causeway over the dead bodies of the advance guard.

Assuming that a Government has decided to continue irrigation developments with subsidies there are a few aspects of policy that might be added to those presented by the Committee.† These fall under three heads: (1) A steady

* *Proceedings*, Am. Soc. C. E., May, 1929, Papers and Discussions, p. 1195.

† *Loc. cit.*, September, 1928, Papers and Discussions, p. 2100.

program of reconnaissance and construction; (2) subsidizing by additional expense for development and settlement in preference to, or prior to, reduction of expenditure on works; and (3) taking the task of rate collection completely out of the range of politics by placing it in the hands of private institutions or banks.

Steady Program.—It is unfortunate that the popular demands for new irrigation developments generally come like sudden devastating floods and they often synchronize with periods of agricultural depression or political elections. They prevent an adequate preparation; they upset all economical arrangements as to staff and construction plant; they cause land to be brought under irrigation before the advent of a steady flow of settlers; and they are apt to disturb market conditions. It is most important that a more or less steady program should be arranged over a long period. Furthermore, there should be no relaxation in the forces of reconnaissance, so that the necessary investigations may go on steadily well in advance of the program of construction. These considerations are more important for irrigation developments than for any other form of public works because essential factors, such as hydraulic and meteorological measurements, can only be properly assessed after long periods of observation.

Subsidizing Developments.—The remarks made by the writer in 1913 as to the slow rate of development and settlement seem to be equally true to-day (1929). If an irrigation project could be built to deal with a slowly expanding demand with the same facility as in the case of railways, roads, telephones, etc., the slow rate of development would not be such a serious factor. Unfortunately, in most irrigation developments, the major part of the ultimate expenditure on works must be incurred at the beginning. A simple study in arithmetic of compound interest and accumulating overhead charges will demonstrate quickly the absolute need of hastening the full development of the land affected by the new irrigation works. Experience has shown that under the existing arrangement (which relies on the private capital of comparatively poor people) development requires at least three times as long as it would with all the necessary capital available at the beginning. Even waiving the interest on capital expenditure for works has not proved sufficient to draw from private sources the capital needed for rapid development on such lands.

If a business that can hardly meet current interest and overhead costs after 20 years can be made to reach the same stage after 5 years, there is a considerable sum (corresponding to lost interest and overhead expenditure during 15 years) that could be utilized better as a free gift in the beginning to accelerate the development. The case becomes even stronger if the slow development of the land under conditions that now exist is not on the soundest possible basis and if the Government, with experienced and trained workers and capital resources readily available, can achieve better development as, for example, in leveling the land, which will result in a greater return from the water made available by the capital expended in the works. It seems preferable, therefore, to give the first subsidy in the form of free capital for accelerating development of the land, but to leave interest charges on at least part of the capital

cost of the works so that its accumulation and excessive overhead costs may be a continual spur toward accelerated development. There will be complications in dealing with those who have already developed land from their own resources and in seeing that an early settler does not take all the advantage at the expense of a later purchaser; but there are complications of this kind in every form of Government bounty.

Collection of Rates Through Banks.—There is nothing so damaging to the financial credit of irrigated areas as the deferment and accumulation of rates that form a first claim on the land. A common failing of certain politicians is to endeavor to gain immediate favor with their electors by exercising the utmost pressure to obtain these deferments. By placing the collections in the hands of banks this kind of political interference will be considerably reduced. Stern action on these lines will also be the best cure for inflated values of land and for the undesirable survival of unfit settlers as well as speculators. These are all very prevalent evils under existing conditions. Moreover, bank management will tend to dampen the activities of those who press for the immediate construction of new projects imperfectly investigated while relying on similar pressure at a later date to evade payment.

JOSEPH JACOBS,* M. AM. SOC. C. E. (by letter).†—It was during the writer's term as Chairman of the Irrigation Division that the Committee on "A National Reclamation Policy" was created, and his letter to the Committee, at the time of its appointment, included the following paragraph:

"In deciding to create this new committee the Executive Committee felt that reclamation matters, particularly in respect of State and Federal relationship thereto, were just now in a state of transition and uncertainty, and that a correct solution of what the governmental attitude should be, required the careful consideration of able men entirely familiar with the subject of reclamation. This is a matter of great economic importance not only to the West and South, where reclamation activities will be greatest, but to the nation as a whole, and it is believed that no other group of men are better equipped, nor indeed anywhere nearly so well equipped, to contribute constructive thought and analyses of these matters, for the advice and guidance of Congress and of State legislative bodies, than is a group of carefully selected engineers, such as is represented by this committee."

That the Executive Committee's apprehension at that time was well founded is evidenced in a growing restlessness concerning reclamation matters and a steadily increasing spread of propaganda directed toward discrediting and discouraging further Federal reclamation activities. So pronounced has this trend been that, on August 26 and 27, 1929, a Western States Governors' Conference was held in Salt Lake City, Utah, to review the situation. However much or little one may agree with the findings of the Committee report, it is gratifying to note that in the solution of important economic public questions which have distinct engineering correlations, engineers specially qualified for the task are willing to contribute their time and their trained judgments to the analyses of such questions, solely for public benefit. This, of course, is as it should be.

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* Cons. Engr. (Jacobs & Ober), Seattle, Wash.

† Received by the Secretary, December 2, 1929.

The writer finds himself in general, although not in complete, agreement with the findings of the Committee. While these findings have been set forth in clear, unmistakable terms, each of them is so important as to have merited some accompanying discussion and supporting data, and the value of the report would have been further enhanced by a prefatory review of the Federal Government's past irrigation activities, its problems, and its accomplishments. Mr. Lippincott, in his subsequent explanatory statement* concerning the report, sensing this curtailment, supplies many of these missing data, and it is to be hoped that the Committee can incorporate some such statement as an integral part of the final report.

The writer cannot accept, without qualification, the implications of Principles (9)† and (10).† It seems to him that the Committee is unduly exercised over the bugaboo of over-production. With a present annual importation of $800 000 000 of foodstuffs, and with a population growth of 1 500 000 per year, over-production in the United States, if it be a fact, cannot long continue and, in any event, it constitutes no sufficient warrant for long delaying the commencement of those major projects which cannot possibly be brought into full yield short of 20 or 30 years. Moreover, agricultural production is not wholly a National problem. It is, to a considerable extent, a regional problem also, and the desirability of an irrigation project the products of which may have, largely, a local distribution, should not be judged too rigidly by National considerations of possible over-production. Local economics must be accorded its due weight in reaching correct final judgments.

Principle (10) declares that the State should share in the responsibility for the selection and approval of projects, and the writer would add that the State should also share in the financial responsibility for the project. State co-operation should be made a condition precedent to the extension of Federal aid. The State's interest in reclamation is greater and more immediate than that of the Federal Government, and second only to that of the settlers on the project. All these agencies should co-operate. If a State, through lack of courage and enterprise, or because it has not sufficient interest or faith in its own development, is unwilling to assume part of the responsibility for that development, it is not entitled to Federal aid. It has no moral right to ask the Federal Government to assume, on behalf of the State, a responsibility which it (the State) is not willing to share. If, in some instances, present State Constitutions inhibit a practical financial co-operation as here contemplated, this can be remedied by Constitutional amendment. A well-planned co-operation should result in more carefully selected projects, in a saner and more rapid settlement, which is all-important, and, finally, in safer project investments for all concerned. If States deem such Federal aid and co-operation of value, they will soon correct any Constitutional defects that stand in the way of their securing it. The formulation of the details of a practical co-operation, of course, will require careful consideration.

* *Proceedings*, Am. Soc. C. E., May, 1929, Papers and Discussions, p. 1193.

† *Loc. cit.*, September, 1928, Papers and Discussions, p. 2099.

Many, no doubt, will take exception to the Committee's condemnation of the waiving of interest on construction charges (Principle (11)*). Some substantial reasons can be advanced for the interest waiver, particularly as part of the original Reclamation Act with its otherwise too drastic payment terms; but the writer believes that the weight of argument, and sound economics, favors the stand taken by the Committee. The only financial concessions that irrigation needs from the Federal Government are a low rate of interest, long terms for payments (not less than 30, nor more than 50, years), and the absorption by the Federal Government of the interest charges accruing against unoccupied lands during the period of settlement. The last-named concession may be regarded, not as a subsidy, but as a payment by the Federal Government for its share of the benefits resulting from irrigation development, and the Federal benefits from such development are by no means inconsequential, whether considered from the social, industrial, or military viewpoint.

With these concessions, any project worthy of development should be able to meet its obligations as they mature. Not only is it the writer's opinion that the projects can pay, and that it is just that they should pay, a low rate of interest on terms as specified above, but that, unless the Western States are willing to accept such a consummation, the Congress will soon refuse approval of any further extension of Federal reclamation beyond present commitments. If such acceptance by the Western States was not justifiable on any higher grounds than expediency, it would have ample, practical justification on that basis alone, because the return to the Government of its investment with a fair rate of interest is probably the only basis upon which land reclamation can be depended as a continuing Federal policy.

The authorities responsible for a National reclamation policy, as Mr. Lippincott points out,† should have in mind the creation of equal opportunities and the stimulation of individual effort; but they should also have in mind that reclamation must be founded on so sound a financial basis that the return of construction costs, plus a reasonable interest charge, is assured. Otherwise, it is not enduring and, sooner or later, must inevitably forfeit Congressional support. It should also be sufficiently broad and adaptive to contemplate the needs of the entire country. People of the West are too prone to interpret reclamation as meaning only the watering of arid or semi-arid lands. From a National viewpoint, however, and this is an all-important viewpoint, the irrigable lands constitute only a small part of the reclaimable lands of the United States. About 25 000 000 acres have already been reclaimed by irrigation, and it is estimated that a like area may yet be served economically by water. The swamp-land area, however, approaches 100 000 000 acres, and the logged-off land area is practically the same. The remaining irrigable area, therefore, represents scarcely 15% of the remaining reclaimable lands.

They of the East and the South, where the major part of these unreclaimed lands lie, are just now (1929), more than ever, voicing the sentiment that the Government's future reclamation policy shall not concern itself solely with arid lands, but shall embrace these other lands as well. It is pointed out that,

* *Proceedings*, Am. Soc. C. E., September, 1928, Papers and Discussions, p. 2099.

† *Loc. cit.*, May, 1929. Papers and Discussions, p. 1196.

whereas the Government has expended millions (about $185 000 000) in aid of irrigation, practically nothing has been done toward reclaiming the swamp and cut-over areas; that the cost of reclaiming these areas would be no more, and generally would be less, than present costs of irrigation; and that they lie nearer the centers of population and, therefore, could be more readily colonized. These arguments appear plausible and may be sound. They merit investigation and consideration in formulating a National reclamation policy.

There has been some suggestion in recent years that the Federal Government confine its activities to the construction of storage works and, on the theory that these are required for flood control, that the Government assume all construction and operating costs, and make the waters thus stored available, without charge, for irrigation and other uses. (This should not be confused with President Hoover's tentative plan, as presented by the Secretary of the Interior at the Conference of Governors at Salt Lake City, on August 26, 1929.) So far as irrigation is concerned, the suggestion is subject to the following objections:

(*a*) In some cases it would be a fictitious basis for Federal aid, for there are districts where there is serious need for irrigation storage and little, if any, need for flood-control storage.

(*b*) In many instances storage may represent but a small part of the development cost. The Columbia Basin Project, for instance, and such projects where magnitude or other conditions impose a necessity for Federal aid, would never be built if the National Government could construct only the storage works.

(*c*) Unless the stored water is charged for (and this charge, if made, should be actual cost) its allocation can become a matter of political favoritism and political spoil. Allocation to various agencies on the basis of priorities, needs, and benefits, and with a fair charge for water on long-time contracts, is more rational and business-like than supplying free storage water. The cost of that portion of the storage works that is properly attributable to needed flood control should be borne by Federal and local governmental agencies; that portion which is necessary for the improvement of navigation should be borne solely by the Federal Government; and those portions which are for the benefit of domestic, irrigation, and power uses should be paid for ratably by those agencies. A fine business integrity is thus preserved, and unnecessary paternalism avoided.

(*d*) To charge nothing for storage water makes the Federal Government figuratively and actually a "wet nurse" to irrigation projects, and that is good neither for the Government nor for the project. A worthy project can, and is willing to, pay for its necessary water supply.

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PAPERS AND DISCUSSIONS

SILT TRANSPORTATION BY SACRAMENTO AND COLORADO RIVERS AND BY THE IMPERIAL CANAL

Discussion*

By Messrs. E. S. Lindley and Thaddeus Merriman

E. S. Lindley,† M. Am. Soc. C. E. (by letter).‡—The observations described in this paper began with an attempt to fix a permanent cross-section; they soon showed its utter instability. Under somewhat similar conditions, the writer found a special application of ordinary rating curves useful.

The conditions referred to are those of the Indus and its tributaries within the Panjab; gradients range from several feet in the mile to less than a foot, in alluvium without any harder formations. Current meters were introduced only during the writer's incumbency, and the observations then available had been made only with surface floats. Supervision had been difficult; observations of higher river stages had generally been impossible; the only good point was that when observations were possible they had been made daily. No individual observation could be accepted as reliable. As a test, as well as for other purposes, it was eminently desirable to try temporary or gradually changing rating curves. In the result, although some of the observations were shown to be utterly unreliable, the majority were very much better than was expected.

A most useful tool was found in plotting the square roots of discharges against gauge-levels; this made each rating curve approximately a straight line, and avoided the difficulty of locating a curve of greatly changing curvature.§

The rating curves of these shifting sandy rivers were found to be much more stable than had been expected. Often a curve held good from the cessa-

* Discussion of the paper by C. E. Grunsky, Past-President, Am. Soc. C. E., continued from January, 1930, *Proceedings*.

† Superintending Engr., Indian Public Works Dept., Panjab Irrig. Branch, Wotton-under-Edge, Gloucester, England.

‡ Received by the Secretary, November 16, 1929.

§ *Minutes of Proceedings*, Inst. C. E., Vol. CCXVII (1923-24), pp. 401-404.

tion of monsoon floods until the winter flood season; there were no data for monsoon flood seasons, but smaller floods affected the curves less than had been expected.

While it was constantly necessary to guard against giving these rating curves undue weight against actual observations, opportunity arose of definitely proving them. About six successive observations at one site were quite consistent among themselves, but disagreed with the curve for the season. They were found to be wrong when discharges passing that site were compared with those above and below. When evaluating losses and gains between successive sites, more consistent results were obtained from daily discharges taken from the curves, than actual daily observations.

This relative stability of rating curves can be understood if it be remembered that an individual silt dune across the river will hardly affect water levels above it, although it will produce higher velocities and a steeper gradient in its own area. The gauge at which a given discharge passes, depends on the "average" bed level for some distance below the gauge site, that average being reached by weighting divergences from the mean grade according to their nearness to the gauge. (It is not suggested that this average can actually be calculated.) Therefore, a mere shifting of silt banks is likely to have no practical effect on the rating curve, and the relative gauge height at which the curve stands from time to time is some measure of the more extended changes of general bed level.

In the observations described in the paper,* an attempt was made to establish a dependable value of Kutter's n by applying the average gradient of an 8 000-ft. length to the average cross-section of a 200-ft. length; but this shorter reach may be one of a number in which the depth is either small with high velocity and sharp local gradient, or great with low velocity and flat local gradient; or it may be one in which velocity is either decreasing or increasing. To determine the value of n dependably, it seems necessary to sound a length of channel sufficient to average these conditions. Friction losses and values of n may depend less on the rugosity as ordinarily estimated from the channel surface, than on the spacing of decelerating sections from rapids to pools which may be considered a large-scale rugosity, and is independent of the former.

In a paper describing observations in Egypt the author, Mr. Arthur Burton Buckley, asserted† a relation between the value of n and the silt charge carried by the flow. In the discussion a number of possible physical explanations for the existence of such a relation were suggested, none of them really satisfactory. A possible explanation that did not then occur to the writer is that the humps and deeps of large-scale rugosity are likely to be more smoothed out in the floods that carry silt. It would be of interest to know whether the observations described in this paper throw any light on this question.

The author quotes a series of analyses which seem to show that the silt along the entire canal system was similar. In 1915 similar observations were

* *Proceedings*, Am. Soc. C. E., August, 1929, Papers and Discussions, p. 1474.

† "The Influence of Silt on the Velocity of Water Flowing in Open Channels," *Minutes of Proceedings*, Inst. C. E., Vol. CCXVI (1922–23), p. 183.

made on the Lower Jhelum Canal in the Panjab, and on some neighboring inundation canals. Samples were collected from just under the surface of silt banks when the canal was not in flow, at a number of points from the head to the tails, outside and inside offtakes. Those of the observations that came under the writer's notice showed fineness increasing regularly with distance from the head. Most offtakes have a tendency to take more than their share of silt, especially the coarser grades; but the pairs of samples at offtakes showed differences insufficient to account for the disappearance of coarser silt; it was not being eliminated by deposition, as there was no steady accumulation of silt anywhere. It was then suggested that even fine silt is subject to attrition, as gravel and shingle are known to be in swifter streams.

The author refers to "boils" on the water surface, and suggests that they are due to the sudden slipping of areas of the river bottom.* The writer believes that he has seen them formed over the paved floors above the under-sluices of Panjab head-works, where it was improbable that the bed silt had any chance of coming to rest in the course of its flow.

The author's reference† to "rhythmic pulsation as if elongated waves were following each other down the river," recalls some points observed by the writer. At the up-stream pier noses of canal bridges there is generally a rhythmic pulsation of water level, up to about 6 in. at some bridges. The pulsations at adjacent piers are of opposite phase, up at one pier when down on the other. The down-stream "wake" of each pier swings from side to side rather sharply, in time with this pulsation. Although the phenomenon did not seem to have any practical importance, the writer once filled some idle minutes by observing it. A pulsation of the same period was observable at the sides of the canal up stream, diminishing with distance, but still noticeable 2 or 3 miles up stream. The water surface showed slight waves; their approximate spacing divided by the approximate mean velocity of flow roughly agreed with the period of pulsation. It had then become too dark to see whether they were moving; they seemed to be on diagonal lines. No observations of the canal bed were made for comparison, but it has been seen to show dunes 6 to 12 in. high in a 5-ft. depth of water, extending across an appreciable part of the width in single lines, and of about the same wave length as the surface waves; but the dunes would only move very slowly, down stream. Rough figures from memory are a 5-sec. period, 15-ft. wave length, and 3-ft. velocity.

THADDEUS MERRIMAN,‡ M. AM. SOC. C. E. (by letter).§—The author has assembled and referred to much of the available data relating to the suspended load carried by the Colorado, but the important question as to how much material this river transports as bottom load is treated only as an incident. Through the literature the writer has long been familiar with this river and a recent opportunity of examining it between Black Canyon and Yuma prompts him to submit the following observations and deductions.

As the discharge of the Colorado increases, a substantial part of the increase in wetted cross-section results from a scouring out of its bottom and

* *Proceedings*, Am. Soc. C. E., August, 1929, Papers and Discussions, p. 1475.

† *Loc. cit.*, p. 1474.

‡ Chf. Engr., Board of Water Supply, City of New York, New York, N. Y.

§ Received by the Secretary, January 6, 1930.

sides. The high-water marks along the river between Black Canyon and Yuma bear mute yet convincing testimony that this under-deepening, at all points of approximately equal width, where the channel is confined by hard shores, is of practically equal volume.

As the discharge increases in the spring of the year the bed of the stream is lowered and widened and the surface rises until a maximum depth is reached at or about the time of maximum discharge. Gauge heights during the flood season on the Colorado vary much more uniformly than on many other rivers because: (*a*) The source of the water is largely in the melting snows; and (*b*) the regulating effect of the long length of river between Glen and Black Canyons tends to iron out the irregularities of flow. By reason of this approximation to uniformity of discharge, long stretches of the river may, at any one time, be said to be at a proportionately equal flood stage. At such a time, therefore, the Colorado channel within any such stretch will have been excavated to sections corresponding to the river discharge. Now if the rate of discharge be maintained for a period sufficiently long for the water to travel, say, from Black Canyon to Yuma, then all the material excavated between these points will have been carried to and delivered below Yuma. The flow records at the Topock and Yuma gauging stations indicate that this condition frequently occurs.

A flood as it advances picks up material in proportion to the velocity of flow and thus the bottom load is increased, while a receding flood is constantly reducing the burden it carries. The quantity of bed material moved out by any flood is equal to the product of its crest length and the area of the deepening induced within that length. At or near the maximum gauge height the river transports a maximum of bottom load, and as long as the gauge height and the cross-section remain constant the quantity transported is the same for each unit of time.

Every flood coming down the Colorado carries in its bilges sufficient material to refill the space which it excavates. The numerical difference between the volume of material removed from any section of the river bed and the volume later deposited therein is always sensibly equal to zero. Were this not so, the regimen of the river above the deltal area would show progressive changes.

Bearing in mind the foregoing considerations, it is possible to predicate, (1) that at low-water stages the bottom load is small; and (2) that as the flow varies, the bottom load in any unit of river length will vary directly with the scoured-out area of the wetted cross-section and inversely as the average velocity of flow.

It thus follows that the scoured out area and the average velocity together constitute a direct measure of the bottom load. That is, the bottom load at any instant in any unit length of the river cross-section is equal to the scoured-out volume of that cross-section divided by the average velocity of flow. Did this equation or condition of equilibrium not obtain, the scoured-out area could never be refilled to its original condition. This fundamental, on which the argument presented is based, may also be stated as follows:

> The bottom load carried per unit of time is equal to the volume of material scoured out per unit of length.

This expression is a simple statement that the continuity of the equilibrium always remains unchanged.

By way of example, if the low-water section area is 3 000 sq. ft. and if this be scoured out by 8 000 additional sq. ft. while the surface rise adds 6 000 sq. ft., then the total section area is 17 000 sq. ft., and the bottom load in any section of unit length is 8 000 cu. ft. divided by the mean velocity, say, 6 ft. per sec. The bottom load thus is 1 333 cu. ft. per unit of channel length and thus 7.8% of the water volume is solid material in motion. As the flow falls off, the condition of equilibrium being maintained, the bottom load is immediately reduced and the bed is partly refilled. Thus if, with a total section area of 8 000 sq. ft., the scoured-out area is 1 000 sq. ft. and the mean velocity is 5 ft. per sec., then 2.5% of the water volume will be bottom load.

The principles advanced should be of ready application, although the writer is aware that certain practical difficulties will arise. In the first place, the Colorado, even at its lowest stages, carries an appreciable bed load and so the zero datum of the scoured-out area may be determined only through a series of approximations; and, secondly, it may develop, on further analysis, that what the writer has designated as bed load may, in fact, be the total load.

The equation of river equilibrium, herein stated, should also find application to a case such as that which the Colorado below Black Canyon will present after the completion of the reservoir at that place. Here the existing equilibrium will be destroyed, and the river will immediately set itself to the task of establishing new conditions of stability. The effects produced as this process proceeds may be forecast by means of the principle of the continuity of the equilibrium.

Accurate knowledge of the total silt load carried by the Colorado is of vital importance in connection with the many developments which center about its waters. The volume of suspended load reported by the observations of the various Governmental departments seems too small to account reasonably for the actual erosion from the 155 000 000 horizontal acres of the Colorado water-shed above Yuma. The conditions on this water-shed are such as to facilitate erosion and the forces at work are great and ever-acting, even under the prevailing small rainfall. It is even conceivable that, in times past, under a greater precipitation, the total erosion may not have been markedly greater than it is under the conditions of to-day. A low rainfall in a barren country may easily be the cause of more erosion than several times that rainfall on an area closely covered with vegetation.

The principles presented, when applied to the records of flow and change in cross-section areas at the Yuma, Topock, Bright Angel Creek, and Lee's Ferry gauging stations, will, it is believed, shed much light on the problem of the bed load which the Colorado yearly delivers to its delta. In any event, the indications are that the volume of this load, when determined, will prove to be substantially greater than the suspended load which is now known within reasonable limits.

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PAPERS AND DISCUSSIONS

NIAGARA POWER
Discussion*

By Lewis B. Stillwell, M. Am. Soc. C. E.

Lewis B. Stillwell,† M. Am. Soc. C. E. (by letter).‡—In this comprehensive paper the author refers to the question: "What led to the adoption of 25 cycles as the standard frequency of alternating current generated at Niagara Falls?" In answer, he quotes§ a paragraph from the Electrical Engineering Papers of the late B. G. Lamme. Mr. Lamme's statement is correct as far as it goes, but further explanation of the origin of the two standard frequencies now in wide use in America, namely, 60 cycles per sec. and 25 cycles per sec., may be of interest.

The frequency originally adopted by the Westinghouse Electric Company in 1885, when that Company actively undertook the development of alternating current for commercial purposes, was 133⅓ cycles per sec. This was the frequency used by Gaulard and Gibbs in demonstrating their system in London, and the original Gaulard transformers imported by Westinghouse were designed for it. A few years later the advantages of direct connection of engine and alternator were clearly recognized, and it became necessary to select one or two lower frequencies adapted to the lower alternator speeds which direct connection implied. In selecting a lower frequency, it was necessary to consider its effect upon transformers, motors, incandescent lamps, and arc lights as well as upon the cost and performance of the alternators. Tesla's American patents covering the polyphase motor had been acquired in 1889 and serious difficulties had been encountered in trying to adapt this motor to 133⅓ cycles.

Experiments demonstrated that American arc-light carbons of that date would not give satisfactory results on frequencies lower than about 60 cycles and after a considerable period of study and experimental investigation, it was decided in 1892 to adopt 60 cycles per sec. as the Company's standard frequency for installations in which incandescent lighting, arc lighting, and motors for industrial purposes predominated. Where a large proportion of the power transmitted by alternating current must be converted to direct current for street railway or storage battery uses, it was decided to use 30 cycles.

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* Discussion of the paper by Norman R. Gibson, M. Am. Soc. C. E., continued from January, 1930, *Proceedings*.

† Cons. Engr., New York, N. Y.

‡ Received by the Secretary, December 16, 1929.

§ *Proceedings*, Am. Soc. C. E., September, 1929, Papers and Discussions, p. 1726.

At the time these frequencies were adopted and, in fact, for many years afterward, converters that would operate successfully on 60 cycles had not been produced. This was the controlling reason which led to the adoption of 30 cycles as an alternative frequency notwithstanding the fact that the engineers of the Company clearly recognized the advantages of a single standard.

In 1893, when formal proposals for the first three 5 000-h.p. alternators and auxiliary equipment were invited by the Cataract Construction Company, the companies submitting tenders were faced by the fact that the hydraulic turbines had been ordered and would operate at 250 rev. per min. As it was found impracticable to design two-phase alternators which, at this speed, would deliver current at 30 cycles, the first formal tender of the Westinghouse Company was for 16 pole machines which would operate at $33\frac{1}{3}$ cycles per sec. Professor George Forbes, one of the consulting advisers of the Cataract Company, advocated $16\frac{2}{3}$ cycles.

In the early autumn of 1893 the writer was asked by the then President of the Cataract Company, Edward Dean Adams, F. Am. Soc. C. E., whether there was any frequency other than $133\frac{1}{3}$ cycles per sec. upon which the Westinghouse Company would be willing to bid. In response the writer expressed the opinion that 25 cycles was practicable. Mr. Adams asked him to take the matter up with his associates at Pittsburgh, and the result was a new tender based upon 25-cycle equipment. This tender was accepted by the Cataract Construction Company, and, in this way, 25 cycles was adopted as the standard frequency at Niagara.

In connection with the first alternators, it was necessary, of course, to design transformers, meters, volt meters, watt meters, etc., for the same frequency, that is, 25 cycles per sec.; thereafter in practically every case in which the Company undertook to supply alternators and auxiliary equipment for a frequency lower than 60 cycles per sec., 25 cycles became the standard.

In this connection, it may be pointed out that the power transmission exhibit of the Westinghouse Company at the World Columbian Exposition in Chicago, Ill., which was designed and constructed during the autumn of 1892 and the following winter, operated at 30 cycles per sec. This exhibit included a 375-kw., two-phase alternator, step-up transformers, a short transmission circuit, step-down transformers, rotary converters supplying direct-current motors and other direct-current apparatus, two-phase induction motors, and arc and incandescent lamps. In every essential, except frequency, it was the system adopted by the Cataract Construction Company. That 25 cycles and not 30 cycles was used at Niagara was due to the fact that the Cataract Construction Company had committed itself to a turbine speed of 250 rev. per min. and to the fact that one of the advisers of the Cataract Construction Company preferred a still lower frequency. In the writer's judgment experience has demonstrated that 30 cycles would have been preferable, especially in view of the fact that at 25 cycles incandescent-lamp service is not always satisfactory. Recognition of this possible difficulty was one of the reasons upon which the Engineers of the Westinghouse Company based their preference for 30 cycles.

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PAPERS AND DISCUSSIONS

FILTERING MATERIALS FOR WATER AND SEWAGE WORKS

PROGRESS REPORT OF THE COMMITTEE OF THE SANITARY ENGINEERING DIVISION

Discussion*

By Charles C. Hommon, Assoc. M. Am. Soc. C. E.

Charles C. Hommon,† Assoc. M. Am. Soc. C. E. (by letter).‡—That part of the report dealing with the selection of coarse-grained aggregates commonly used in sewage trickling filters is a rather thorough treatment based on data secured by laboratory tests.§ The writer has just completed an extensive study and report based upon the actual performance of forty-eight sewage filters in the States of Michigan, Indiana, Ohio, Pennsylvania, Alabama, Tennessee, Massachusetts, New York, New Jersey, and Georgia. The information secured through this investigation brings to light some items that are not in accordance with the findings of the Committee.

The importance of durability of aggregates used in sewage trickling filters is well recognized by all sanitary engineers, and this item has been protected in the past to the fullest extent by writing many safeguards into specifications. The studies by the writer brought to light many sewage filters, however, that were filled with improperly graded aggregate. The question that naturally arises is one of durability. Was the aggregate of proper size and grading at the time of initial construction or had disintegration been responsible for the improperly graded material as found?

In the case of some of the older filters it is impossible to answer the question definitely, or to state what percentage of the material had been affected by disintegration. However, the weight of evidence strongly indicates that

* Discussion of the Progress Report of the Committee of the Sanitary Engineering Division on Filtering Materials for Water and Sewage Works, continued from January, 1930, *Proceedings*.

† San. Engr., in Chg. of Sewage Treatment, Municipal Sewage Treatment Plant, Canton, Ohio.

‡ Received by the Secretary, December 14, 1929.

§ *Proceedings*, Am. Soc. C. E., September, 1929, Papers and Discussions, p. 1801.

in most cases the unsatisfactory condition of the aggregate is due either to the fact that proper sizes and grading were not stated in the specifications or to failure in securing the sizes specified. With few exceptions, inspection of the existing materials after a period of years of service indicates that properly graded material placed in the beds at the time of construction would have provided sufficient tolerance to care for the slight quantity of fines produced through disintegration. The chemical and physical analyses of materials collected at the time of inspecting the various filters were of little value in determining the weathering abilities of the respective materials. Even in the few filters where disintegration was known to have been most active, a study of the chemical and physical data shed but little light on the subject. The limiting percentages placed on calcium, magnesium, and iron oxides and sulfur in blast-furnace slags, as set forth in the report (see Table 1*) are not in accordance with the data secured by the writer. The sodium sulfate test carried to twenty cycles appeared to be the most definite indication of the ability of the aggregates to withstand the destructive effects of weathering. This test, as applied to the samples collected, did not register in all cases the minor effects of weathering, such as flaking and occasional splitting, which were noted in the field investigation. In some instances, however, the sodium sulfate test did approximate the conditions found in the field by fracturing some of the pieces tested, although few of the samples tested could have been adjudged unsound material.

The comparative unloading ability of rough aggregates, such as blast-furnace slag, and more or less smooth material, such as limestone, granite, trap-rock, and gravel, was carefully noted throughout this investigation. After observing a number of different aggregates, the final conclusion was that there is but little difference in their unloading abilities. Since filters composed of granite, trap-rock, feldspar, limestone, and gravel are known to have become excessively dirty and sometimes clogged to the extent of making necessary the removal of the aggregate for cleaning, one is forced to the conclusion that automatic unloading is not dependable in all cases and that surface texture plays but a minor part in retaining the deposited solids. To determine the feasibility of controlling the deposits normally collected in sewage filters an attempt has been made to force the unloading of the Canton, Ohio, filters as desired.

Prior to the experiment the filters had been in service for approximately one year, during which time some sloughing had occurred, but not sufficient to prevent some slight surface ponding due to algæ growths. The slag medium was quite dirty to a depth of about 6 in. First, some chlorine that was being added to the raw sewage in connection with some other investigations, caused a very decided sloughing of the organic deposits. Following this partial unloading, a section of the filter area was cut out of service and rested. When the dried area was placed back in service a very decided unloading of the organic deposits was noted.

With the coming of warm weather during 1928, it became necessary to flood the filters in order to control the development of the filter fly. It was

* *Proceedings*, Am. Soc. C. E., September, 1929, Papers and Discussions, pp. 1812–1813.

noted that the flooding also caused a very marked sloughing, the suspended solids in the effluent amounting to as much as 800 parts per million for short periods of time. Throughout the summer, flooding and resting were continued with the result that by mid-summer the filtering material was practically as free of organic deposits as when originally placed in the beds. The analytical data indicated that the quality of the effluent was not impaired in any way, but perhaps had a more constant stability. Over a two-year period the nitrate nitrogen has been increased from zero to about 12 parts per million, while the bio-chemical oxygen demand has been reduced about 80%, or, from 108 to 22 parts per million.

A number of test pits dug to varying depths ($1\frac{1}{2}$ to 7 ft.) in the respective units indicated that the cleansing action extended throughout the filter. Recent repair work exposed a considerable portion of the material for its full depth in a part of one bed. This showed very clearly that the sloughing had been effective in the bottom areas as well as at the top, and, further, that none of the deposits had lodged in the under-drainage system. When compared with the amount of organic deposits observed in many filters located generally over the country, the cleanliness of the Canton filters leads one to conclude that the quantity of deposits retained in the respective beds is due to other causes than surface texture of the filtering material.

Reference has been made herein to the need for guarding against material having cementitious qualities, such as some blast-furnace slags. It is well recognized that limestone as well as blast-furnace slag will consolidate in the presence of water if pulverized sufficiently fine. However, the amount of fines required to cause solidification of a coarse aggregate is sufficiently large to preclude the use of such poorly graded material in sewage trickling filters. The fact that such an improperly graded aggregate would soon become clogged by organic deposits (which, in practice, would be similar in effect to solidification), renders the item of cementation of but little importance. Thirty-seven of the forty-eight sewage works inspected by the writer used crushed blast-furnace slag as a filtering medium and evidence of cementation was looked for in all filters observed.

Only two instances were found and one of these involved a very small area of the bed. In the second case, a previous investigation estimated that approximately 20% of the aggregate in the filter had been affected by cementation. In both cases the cementing action had been caused by excessive fines deposited in localized areas through improper handling of the aggregate at the time of placement. An excellent opportunity for observing the cementing qualities of such material was afforded at the time of reconstructing the Canton sewage works in 1926, when the old contact beds were dismantled. All the aggregate was removed from the contact beds and careful note was made of the condition of the slag, but the engineer in charge of the work reported that there was no evidence of cementation.

Perhaps one of the most enlightening features of the writer's investigation was the relatively wide range in the size and grading of aggregates. The lack of uniformity was not only apparent when comparing one plant with another, but often occurred within the same filter. Such a condition indicates a lack

of crystallized opinion among sanitary engineers regarding proper sizes and grading.

One of the most essential features upon which the sewage trickling filter is founded is that it must afford a ready passage for sewage and air at all times. Not only maximum percentage of voids but large-sized voids are very necessary; therefore, proper sizing and grading are exceedingly important. The shape of the aggregate after being crushed is also important, angular and cubical-shaped material being preferred. Such shapes prevent nesting, and through their relatively large-sized voids they afford an easy and quick passage for the removal of the larger particles of organic deposits from the filter during unloading periods.

The surface texture (degree of roughness) is an item that has been debated by the Sanitary Engineering Profession for several years. The additional surface area afforded by the roughness is quite generally admitted to be an advantage, but some engineers are inclined to believe that the roughness may have a tendency to prevent free unloading of the organic deposits. This item was given careful thought during the field investigation, and, whenever possible, comparison of different materials was made. The many variables, such as loads, climatic conditions, method of operation, and character of sewage, necessarily made it difficult to compare one filter with another in definite terms.

The fact that the vast majority of the older filters (regardless of type of filtering material) were quite dirty, together with the service record of some of the oldest filters which were filled with blast-furnace slag and were still functioning, forces one to conclude that surface texture played but little or no part in the removal of organic deposits from the beds during the unloading periods. The slight difficulty experienced in keeping the slag filters at Canton practically free from organic deposits, and the very satisfactory degree of treatment afforded the sewage, raises the question as to possible superiority of a rough material over a smooth or semi-smooth aggregate. Because of dissimilar conditions, this question cannot be answered by comparing the aggregates in different plants. It can only be determined through the use of test units on which the different materials may be subjected to the same sewage under conditions that are alike in all respects.

Perhaps one of the most annoying features of a successfully operating sewage trickling filter, especially during the summer months, is the presence of so-called filter flies (*Psychoda alternata*) which develop in great numbers under favorable conditions. During this investigation an effort was made to determine whether the character of the filtering medium had any effect on the development of the fly, but the type of aggregate did not appear to play any part. The development of the fly in sewage filters may be called a normal occurrence, and flies were found in greater or less numbers in practically all filters visited. Data secured in operating the Canton plant indicate that the quantity of organic deposits retained in the medium is perhaps the controlling factor.

During the summer months the development of flies in the Canton filters was the most active the writer has ever observed. Throughout the breeding season it

has been necessary to flood every ten days, and this has been very effective in controlling the adult fly. Since flooding is not only effective in drowning the adult specimens, but also arrests or stops the development of the larvæ, it is only fair to assume that a filter which is sufficiently dirty to cause surface pooling is effective in controlling the release of the adult specimens, somewhat in proportion to the quantity of water retained in the filter. In other words, it appears that a medium which is practically free from deposits and, therefore, does not retain any appreciable quantity of free water will breed the greatest number of flies; likewise a dirty filter will develop the least number. The almost complete absence of the filter fly at some of the more seriously clogged filters observed during this investigation adds to the belief that the development of the fly is dependent upon the amount of organic deposits retained in the filter rather than the type of the filtering material.

The Committee is to be complimented upon the mass of data which it has collected and compiled, as well as for the extent of the labors represented by the report. However, from the writer's personal investigation and study it would seem that a number of properties of filtering media to which considerable weight has been given heretofore are of only relative and indefinite importance. This refers to such properties as weight, specific gravity, chemical composition, absorption, and abrasion. Limitations on these properties tend toward a hypothetical product which in many localities may prove costly. It would seem that four qualities supersede all other considerations in importance: (1) Proper sizing and gradation; (2) shape of particles and size of voids; (3) durability for which the best present determination is the sodium sulfate test carried to 20 cycles using at least fifty pieces; and (4) local costs cannot help but be an important consideration.

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PAPERS AND DISCUSSIONS

SPILLWAY DISCHARGE CAPACITY OF WILSON DAM

Discussion*

By J. H. Jones, Esq.

J. H. Jones,† Esq. (by letter).‡—Whether or not one agrees with the conclusions drawn it is always an instructive pleasure to read a carefully carried out investigation, and the equally careful analysis, such as Mr. Puls has made. In carrying out his velocity measurements on such deep streams he has placed all hydraulic engineers in his debt, and British engineers who have few opportunities of dealing with large volumes of water find such researches especially valuable.

This discussion is not intended to present criticism, but an alternative means of attacks which may be of interest. The method has been crudely suggested in some notes by the writer, entitled, "The Standing Wave."§

Briefly, it consists of a denial of the usual theory, that, in the discharge through a large orifice, the velocity of any filament at a distance, z, below the free surface (see Fig. 4‖) is proportional to $\sqrt{z}$. Throughout the full depth at the *vena contracta* the velocity is sensibly uniform and proportional to $\sqrt{h_2 - cD}$, in which,

h_2 = depth from free surface to bottom of orifice.
c = coefficient of contraction.
D = depth of opening of gates (orifice).

The *vena contracta* is considered, since only there is the velocity of all filaments purely axial. Hence, the discharge from an orifice of length, L, depth, D, and with a head, H, is:

$$Q = c\,D\,L\,\sqrt{2g\,(H - c\,D)} \quad\quad (13)$$

* Discussion of the paper by Louis G. Puls, Assoc. M. Am. Soc. C. E., continued from January, 1930, *Proceedings*.

† Chartered Civ. Engr., Kilmarnock, Ayrshire, Scotland.

‡ Received by the Secretary, November 20, 1929.

§ *Engineering*, August 10, 17, and 24, 1928; see, also, "Notes on Sluice-gate Discharge," *Engineering*, September 29, 1929.

‖ *Proceedings*, Am. Soc. C. E., October, 1929, Papers and Discussions, p. 2135.

The value of c is difficult to establish, but, in a sluice-way with a horizontal bed, it does not differ greatly from two-thirds. However, in the present case, because of the distortion caused by the blunt piers, 0.62 is the value used. Solving Equation (13) for various openings, D, under a constant head, $H = 18$ ft., on a 38-ft. gate-opening, the results are as listed in Table 4.

TABLE 4.—COMPARISON OF DISCHARGE COMPUTED FROM TWO EQUATIONS.

D, in feet.	Q, IN CUBIC FEET PER SECOND.	
	By Equation (7).	By Equation (13).
1	1 039	785
2	1 522	1 540
3	2 214	2 260
4	2 958	2 960
5	3 640	3 630
6	4 325	4 255
7	4 990	4 860
8	5 565	5 450
9	6 080	5 980
10	7 510	6 460
11	8 260	6 930
12	8 960	7 350
13	9 925	7 760

It will be noted that these only differ appreciably from the values obtained by Mr. Puls' complex equation as given in Table 1,* in the highest values of D; but here the reduction in c, due to the piers and grooves, is least, and the *vena contracta* is less definite, and the values of Q are thus greater than those calculated.

The value of Q for $D = 1$ ft., given in Table 1 as 1 039 cu. ft. per sec., does not agree with Fig. 7,† which is about 800, and it would appear that the sign and nature of the deviation of the writer's calculated discharges from those of Mr. Puls' formula, agree closely with the deviations shown by the readings on the Florence gauge.

Those who have facilities for sluice-gate measurements should try out this method as a check, so that engineers may possibly, as the information accumulates, be able to form some method of correlating values of c with the detail design of sluice-ways.

* *Proceedings*, Am. Soc. C. E., October, 1929, Papers and Discussions, p. 2138.
† *Loc. cit.*, p. 2140.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

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THE BEHAVIOR OF A REINFORCED CONCRETE ARCH DURING CONSTRUCTION

Discussion*

By Messrs. Herbert J. Gilkey and A. H. Fuller

Herbert J. Gilkey,† M. Am. Soc. C. E. (by letter).‡—In this interesting paper there are two or three points that suggest questions.

The top curve of Fig. 7§ shows a concrete unit compressive stress of nearly 2 000 lb. per sq. in. in the crown concrete at intrados (surface measurement). The age of this concrete at that point on the curve appears to be ten days, or less. Notwithstanding the rather strong mixture used (as indicated in Table 1‖) a stress of 2 000 lb. per sq. in. approached the ultimate strength at this age rather closely. Unless a secant modulus for a correspondingly high stress was used in the reduction of strains to unit stresses, it is probable that a stress considerably less than the 2 000 lb. per sq. in. shown corresponds to the high measured strain. Similar comment may be applicable to some of the other high concrete stresses if the proportional limit of the concrete was exceeded.

In Table 4¶ the great range of values for coefficient of expansion for the deck concrete (from 0.000004 to 0.000056 per degree centigrade) is such that these data should probably not be considered as measures of thermal expansions at all. As the author states,¶ and as is clearly shown in the "Weather" column of Table 4, it is probable that volumetric changes due to moisture (wetting and drying) had more effect than temperature. This is on the assumption that the measurements give correct indications of the extent of volumetric change that occurred.

* This discussion (of the paper by Searcy B. Slack, M. Am. Soc. C. E., published in November, 1929, *Proceedings*, but not presented at any meeting of the Society), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

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‡ Received by the Secretary, December 2, 1929.

§ *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2291.

‖ *Loc. cit.*, p. 2284.

¶ *Loc. cit.*, p. 2298.

There appears to be a slight discrepancy between Conclusion 6* and Fig. 13† (lower curve). The temperature appears to be rising for at least six days after pouring while Conclusion 6 states that the temperature starts to fall after 12 to 36 hours (from 0.5 to 1.5 days). While the apparent discrepancy is not great, the question is rather basic and the reader should know which of the evidence to accept.

Concrete is described† as a nominal 1:2:4 mix, the quantity of water being carefully gauged to give a concrete with a slump of from 3½ to 4½ in. Measurements were by loose volume, but the bulking of the fine aggregate was determined twice daily and a correction was made in the mix for the bulking.

Evidently, the correction applied solely to the volume of the fine aggregate since the only apparent effort to regulate the quantity of water used was based on slump determinations. It is unfortunate if this is true. As is now well known, small variations in the water content have a much greater influence on the properties of concrete than small variations in either the quantity or grading of the aggregate. Much more uniform concrete would have resulted had the water-cement-ratio been kept constant and the quantity of aggregate varied to maintain constant slump.

The arch rib itself is a large member and its action is the resultant of an assemblage of parts which are less uniform than they might have been. The few auxiliary samples used represent only a very limited part of the concrete in the structure and there is a question whether the strengths and moduli of elasticity found approximate the mean values for those of the structure as a whole. It is well known that ordinary stock-pile variations in grading will introduce important variations in the water requirement for a constant slump.

At its worst, however, the indications can probably be accepted as qualitatively correct if not quantitatively refined. Moreover, they are probably as well controlled as the test results from other similar projects. Well controlled experimental work is still difficult to attain in connection with the building of a large structure.

A. H. Fuller,‡ M. Am. Soc. C. E. (by letter).§—The tests upon which this paper is based add much to the knowledge of the behavior of concrete structures. The author is to be congratulated upon the care with which they have been made, recorded, and presented. He modestly states‖ that they are not conclusive, but that they indicate phenomena well worthy of further study. The first step in the study is naturally a critical review of this paper and others that deal with the same subject. The writer will confine his discussion to Conclusions 2¶ and 4¶ and, in doing so, will accept, as within recognized limits of precision, all statements of fact throughout the paper. His experience leads him to make interpretations of the facts upon which Conclusions 2 and 4 are based, which differ from those made by the author.

* *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2297.

† *Loc. cit.*, p. 2283.

‡ Prof. of Civ. Eng. and Head of Department, Iowa State Coll., Ames, Iowa.

§ Received by the Secretary, December 9, 1929.

‖ *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2280.

¶ *Loc. cit.*, p. 2299.

In Conclusion 2 the author states that, in the design of concrete arches, the allowance for temperature stresses should be based entirely upon a fall in temperature. The underlying facts are that the set in the concrete takes place under high initial temperature and that considerable strength is developed in the concrete before the temperature drops to normal. The conclusion would be justified if the materials were truly elastic; but concrete is not truly elastic. Available data demonstrate that concrete is plastic to a considerable extent. They also indicate the extent of plasticity or "flow of concrete" or "time-yield" under a few conditions.

In 1915 the writer, with C. C. More, M. Am. Soc. C. E., was confronted with the task of interpreting the stresses in the Bell Street Warehouse, in Seattle, Wash., where deformations in several parts of the building—which showed no indications of distress—far exceeded the deformations in the control cylinders at impending failure. The next step in the interpretation of the stresses was to observe three other control specimens under continuous load in which the successive unit stresses developed and the elapsed time under each load corresponded fairly well with conditions in the building. The results gave a fairly satisfactory basis for interpreting the stresses in the building. They also indicated the amount of the "time yield," for the particular material.[*]

A study of this and other researches on the plastic flow of concrete[†] indicates continuous deformations in concrete under unchanging but continuous load which, for the same time and for similar stresses, exceed the deformations which would be produced by the drop in temperature which the author noted in the Eatonton arch.[‡] The writer interprets these data to indicate that, as the temperature dropped in the arch, the slow deformations produced gradual re-adjustments in stress rather than constantly increasing stresses. It seems likely that these re-adjustments finally resulted in the practical elimination of temperature stresses under normal temperature. It also seems likely that at least partly complete re-adjustments will take place. These will reduce greatly the temperature stresses from seasonal changes, leaving the definite temperature stresses confined to those resulting from temperature changes within a reasonably short time.

In Conclusion 4, the author states that observed stresses in the steel and the concrete were consistently higher than the computed stresses. The curves[§] in Figs. 6, 7, and 8 confirm the statement. These curves indicate stresses rather than deformations and are, therefore, dependent upon the interpretation of facts rather than upon the facts themselves. The author states[||] that:

"Until more information is available on stresses in steel caused by the setting phenomena of concrete, a fair comparison of computed stresses with measured initial stresses cannot be made."

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* *Proceedings*, Am. Concrete Inst., Vol. 12 (1916), pp. 302–310; *Proceedings*, Pacific Northwest Soc. of Engrs., Vol. 15 (1916), pp. 7-57.

† "Researches in Concrete," W. K. Hatt, M. Am. Soc. C. E., *Bulletin 24*, Eng. Experiment Station, Purdue Univ., Lafayette, Ind.

‡ *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2286.

§ *Loc. cit.*, p. 2291.

|| *Loc. cit.*, p. 2292.

The writer not only agrees with the statement, but would extend it to include measured stresses in concrete. He believes that existing data have decided value in interpreting the deformation of concrete and in pointing the way for further research for which the author justly asks.

To refer again to the Bell Street Warehouse, it was apparent, in deducing the stresses in concrete, that the modulus of elasticity, as a measure of the relation of stress to deformation while the load was being applied, was of doubtful value in interpreting the stresses that were represented by the observed deformations in the building. However, the modulus of elasticity, as a measure of the relation of stress to deformation as determined from time tests (in which the rate of application of the load was about the same as in the building), did furnish means for making a fairly satisfactory estimate of the existing stresses.

It will be noted from Figs. 6, 7, and 8, that, while the observed stresses in the steel usually exceeded the computed ones as was stated, the two were approaching each other at the end of the period of observation. Inasmuch as the translation of deformations into stresses in steel cannot be questioned, and as the paper has demonstrated that the initial stresses in the steel were increased by the setting of the concrete, the writer believes that, as the concrete was adjusting itself during the four months under observation, the stresses in both the concrete and steel were approaching normal values.

The Engineering Profession is deeply indebted to Mr. Slack for demonstrating the points which he has given under Conclusions 1, 3, 5, and 6.* If his request for further study should result in an extensive research which would permit definite interpretations of the matter in Conclusions 2 and 4, the debt would be still greater.

* *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2299.

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PAPERS AND DISCUSSIONS

FACTORS GOVERNING THE LOCATION OF AIRPORTS

Discussion*

By George D. Burr, Assoc. M. Am. Soc. C. E.

George D. Burr,† Assoc. M. Am. Soc. C. E.—The subject of the proper location of airports is a timely one. It is perhaps too early to attempt to assign definite values or weights to the factors that must be taken into consideration in the intelligent selection of an airport site.

There is only one thing that commercial air transportation has to sell and that is speed, or time saving. The one factor on which the sale of this commodity hinges is safety and certainty of arrival on time. Therefore, a commercial or municipal terminal airport must be located as close to the metropolitan center of the city as safety will permit.

This rule is quite simply stated and quite difficult to apply. However, with existing commercial types of heavier-than-air craft, no site will be satisfactory regardless of other conditions: (1) If meteorological conditions are unsafe; (2) if the site is too small for safe landings or take-offs; (3) if dangerous flying practices are permitted over, on, or near the field; or (4) if the field is a long-time distance from the ultimate termination of the journey of passengers or commodities.

The cost of purchasing land, bringing utilities to the site, developing transportation facilities, improving difficult drainage, surfacing runways, leveling land, and similar factors, might be grouped under one single head, namely, cost of site.

If the nearest safe site that may be developed economically is a relatively great distance from the ultimate destination of passengers traveling to the city served it might be used, but always with the thought in mind that, although it is an aid to air navigation, another site, closer to the final destination, will some day be demanded when air transportation has developed sufficiently to make such more expensive sites economically feasible. However, if a site is close in, but is decidedly unsafe, because of meteorological conditions or small area, that site will be abandoned in the future.

* Discussion of the paper by Donald M. Baker, M. Am. Soc. C. E., continued from January, 1930, *Proceedings*.

† City Traffic Engr., San Francisco, Calif.

These considerations would leave only three items to be considered in the selection of a terminal airport as follows:

Factor.	Weight.
(*a*) Safety, with consideration of fog, smoke, wind, turbulence of air, and safe area of field	Mandatory.
(*b*) Distance from ultimate destination	As close as feasible in consideration of safety.
(*c*) Cost	Select the site that is cheapest to develop if two or more sites present the same relative advantages from the point of view of Factors (*a*) and (*b*).

It must be borne in mind, however, that these remarks appertain to the selection of a terminal commercial or municipal airport for heavier-than-air craft only. There are many more classes of airplane landing fields to which these principles will not apply. For instance, there are: (1) Emergency and intermediate fields placed along an airway; (2) fields developed for the sole purpose of providing a training place for students; (3) fields located primarily as a junction and transfer point for air mail lines with little local delivery; (4) fields used principally for some specific commercial purpose, such as for tests by an airplane manufacturer, or as a terminal point for an air mapping company, or an air "taxi" or "joy hop" recreation center; and (5) fields devoted almost exclusively to the storage of privately owned craft used for private business or pleasure. The present tendency is to specialize in the use to which a field is to be put.

The question of just what amount of money may be justifiably expended on a given site by a given community or commercial enterprise is a matter for definite analysis. Eventually, the airport must pay a return on the capital invested.

There are several sources of revenue from a publicly owned airport enterprise. These are rental of hangar space; rental of concessions, such as restaurants, gas stations, etc.; rental of office space; charges for use of special facilities; and charges for landings. At present, charges for landings are seldom imposed, in order that flying may be encouraged. Hangar rentals and office rentals form the principal, immediately available, source of revenue.

A charge of 3½ cents per sq. ft. per month, and sometimes more, for hangar space is now quite generally imposed. Hangar space may be constructed for approximately $2.50 per sq. ft., including all costs, exclusive of land. If such hangars bring a return of 2½ cents per sq. ft. per month, an adequate return is made to amortize the investment in the structure within a reasonable time, and to pay interest, repair, and depreciation charges. The remaining 1 cent per sq. ft. per month of return may go to help pay field operating expenses, maintenance, and purchase cost of land. This specific feature has been elaborated somewhat in order to show the nature of the financial problem.

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PAPERS AND DISCUSSIONS

HIGH DAMS

A SYMPOSIUM

Discussion*

By Messrs. Fred A. Noetzli and Edward Godfrey

Fred A. Noetzli,† M. Am. Soc. C. E. (by letter).‡—In the design and construction of dams of ever-increasing height many new problems are encountered, some of which are of vital importance for the safety of such structures. If the law of similarity would hold in every respect, then, within proper limits, high dams built on an adequate foundation should be as safe as low ones.

Unfortunately, certain conditions have the tendency of making high dams relatively less safe than lower structures of the same type. Attention is called, in this connection, first, to the danger of buckling the face slabs of high earth or rock-fill dams due to a relatively larger settlement of the fill of the higher dams; second, to the complicated stress conditions and the tendency of tension cracks developing in the thick arches of high arch dams; and, third, to the tendency of cracks developing in the buttresses of high dams of the buttressed type, such as multiple-arch, Ambursen dams, etc. Only recently it was observed that shrinkage cracks have also occurred in the interior of high concrete gravity dams. While such interior cracks may not be serious in case of relatively low gravity dams in which the shear stresses are small, the stability of high gravity dams may be seriously endangered if the cracks should extend in the direction of high shear stresses.

Large concrete dams of the gravity type, if built rapidly, develop high temperatures in the interior of the mass. Later, when the internal heat is being dissipated, the concrete shrinks and, being restrained along the rock foundation, is setting up tension stresses which may exceed the tensile strength of the concrete and produce cracks. In order to avoid such cracks, vertical contraction joints, usually about 50 ft. apart, are being provided in practically all

* Discussion of the Symposium on High Dams continued from January, 1930, *Proceedings*.

† Cons. Hydr. Engr., Los Angeles, Calif.

‡ Received by the Secretary, November 29, 1929.

concrete gravity dams. Not until after the failure of the St. Francis Dam was the definite proof available that similar cracking was likely to occur also in planes parallel to the axis.

In Fig. 7 are indicated some of the cracks which were observed in the remaining part of the St. Francis Dam after the failure. Water seeped out for several days after the failure, indicating that the cracks extended over considerable areas and were subjected to internal water pressure. A gravity dam, say, 250 ft. high, may be about 170 ft. thick at the base. There is no reason why the concrete should not shrink in an up-stream and down-stream direction the same as in the direction of the axis of the dam. Thus, in the 170-ft. thickness of the dam, the shrinkage of the concrete will probably produce cracks unless contraction joints are provided at suitable intervals. There is no other alternative unless new materials or construction methods are developed which will furnish concrete that does not shrink. As Mr. Henny pointed out,* "such cracks are likely to be approximately vertical and are thus often close to the direction of maximum shear."

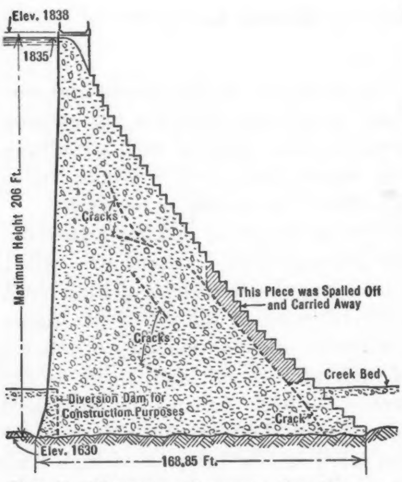

FIG. 7.—MAXIMUM SECTION, ST. FRANCIS DAM, SHOWING CRACKS.

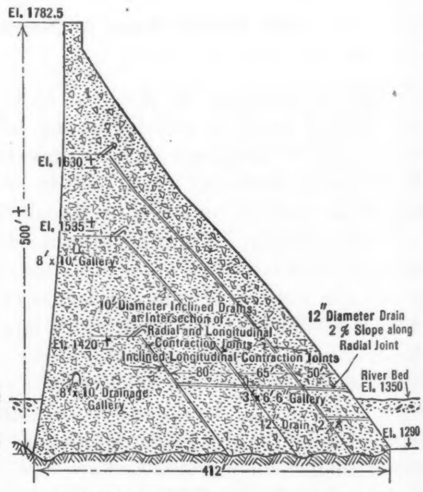

FIG. 8.—MAXIMUM SECTION, SAN GABRIEL DAM, SHOWING INCLINED JOINTS.

Fortunately, the theory of the principal inclined stresses offers safe and adequate means for avoiding irregular internal shrinkage cracks by providing inclined contraction joints parallel to the direction of the principal stresses for maximum load. In these planes the shear is zero. Consequently, there is no tendency for a relative movement to take place in the parts of the dam on opposite sides of such joints. Contraction or expansion of the concrete may take place by opening or closing of the joints and with little or no effect on the load stresses in the dam.

Fig. 8 shows the location of the three longitudinal joints in the cross-section of the San Gabriel Dam. The joints are offset in the sections on oppo-

* *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2334.

site sides of the vertical radial contraction joints. Interlocking keyways will prevent excessive movements of any portion relative to the adjoining portions so that the whole structure will act as a system of inclined columns, each of which is thoroughly interlocked with the adjoining columns. The inclined columns have a horizontal cross-section of approximately 50 by 60 ft. They may be built up individually for several lifts ahead of the adjoining columns so that at least a part of the shrinkage can take place before others are poured to the same elevation. Also, the individual pours may be interlocked with each other. In this case the inclined joints would have offsets of perhaps 6 to 12 in. each at intervals of, say, three or four lifts each. If these offsets are troweled smoothly and painted with oil to prevent bond, a slight lateral movement corresponding to the shrinkage of the concrete could take place and thus the inclined joints made effective without material sacrifice of the strength of the structure or of the effectiveness of the inclined joints.

In this connection attention is called to the San Mateo (concrete gravity) Dam built in 1887 and 1888 for the water supply system of San Francisco, Calif. This dam was constructed of blocks of concrete, 10 to 12 cu. yd. each, cast in place and thoroughly interlocked with each other. If the theory of the principal inclined stresses in gravity dams had been known at that time, and if the designer had arranged the up-stream and down-stream slopes of the individual blocks in continuous lines parallel to the direction of the principal stresses, this dam would be representative of the most advanced type of construction. Apparently, the designers and builders of the San Mateo Dam anticipated by sheer ingenuity or intuition a requirement in the construction of concrete gravity dams which only recently was substantiated by theoretical deductions. It can no longer be ignored without inviting possible disaster, especially in the case of high dams.

Similar conditions as to shrinkage cracks obtain in the buttresses of high multiple-arch and Ambursen dams. . Cracks have been observed in the buttresses of many of these structures. Fortunately, in most cases they appear to have developed under load conditions, and their direction was guided by the laws of Nature to be approximately parallel to the principal stresses. Two engineering commissions recently investigated the Lake Hodges Dam in California. Both reported that the various inclined cracks, of which there is at least one in practically every buttress of the dam, did not render the dam unsafe. This conclusion seems to be substantiated by the fact that the Lake Hodges Dam, built in 1917, has since been filled several times. The buttresses contain practically no steel reinforcement; nor is any reinforcement apparently contemplated despite the presence of the cracks, some of which are nearly 0.1 in. wide and extend from the base of the buttresses forwardly inclined to practically the groin line of the arches.

The cracks in the Lake Hodges and other dams are evidently a good and sufficient proof of the feasibility, not to say, necessity, of providing large concrete dams, both of the buttressed and of the gravity type, with inclined contraction joints. The agreement of these "full-size tests" with the theory of the principal stresses is quite satisfactory and is further subject to verification by constructing in the ordinary way the force and string polygons for water load

and weight of the individual parts of the dams separated by the joints. This was done in the designs of the Coolidge (multiple-dome) Dam, completed in 1929 for the U. S. Department of the Interior, Indian Service, on the Gila River, in Arizona, and for the San Gabriel Dam.

Mr. Wiley's statement* that, in his opinion, "the higher dams of the future will be of a solid masonry gravity type," is somewhat startling. As a matter of fact the two highest dams built so far, namely, the Diablo Dam on the Skagit River in Washington, 385 ft. high, and the Pacoima Dam in California, 372 ft. high, are of the single arch type. There is under construction in France an arch dam which, when completed, will be 440 ft. high. This dam is located in the extremely narrow canyon of the Drac River in the French Alps and is only 55 ft. thick at the base.

The Owyhee Dam, 405 ft. high, mentioned† by Mr. Wiley as being of the arched gravity type, was designed and is being built as an arch dam. Furthermore, studies are being made to determine whether the proposed Boulder Canyon Dam, 680 ft. high, can not be designed to act as an arch, at least in the upper portions, thereby decreasing the enormous quantity of concrete required for a pure gravity dam while at the same time increasing its safety factor against overturning and sliding. Thus, it is seen that the highest dams so far built or under construction are not of the gravity but are of the single arch type.

Dams of the buttressed type, after cautious beginnings some time about 1910, have already been built to a height of 250 ft. There is no reason why dams of this type should not be built with perfect safety to still greater heights. Undoubtedly, this will be done sooner or later.

A gravity dam has a true factor of safety of less than two. This is less than for any other type of engineering structure. Furthermore, such a dam is subject to uplift pressure and to internal tension stresses of unknown, but certainly large size, sometimes resulting in cracks such as have been observed in the interior of a number of dams of this type. While these cracks appear to be of small consequence in case of relatively low dams, because the shear stresses are small, they are of increasing importance for high dams especially if, as Mr. Henny pointed out,‡ the cracks should occur in planes of high shears.

It is generally believed that the design of a gravity dam is a simple matter and permits of an easy determination of the stresses. Recent investigations have shown, however, that a scientific design of a high gravity dam is a fairly difficult problem; at the same time it involves relatively large uncertainties, especially as regards the applicability of the law of the trapezoid for the stresses in thick dams, uplift pressure, internal temperature, and shrinkage stresses, etc. While some of the same difficulties are encountered in the design of high buttressed dams, the tension stresses in the buttresses may be taken care of by steel reinforcement; a similar procedure is not economically feasible in the case of gravity dams.

* *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2318.

† *Loc. cit.*, p. 2319; *Western Construction News*, January 10, 1929, p. 9.

‡ *Loc. cit.*, p. 2334.

From these considerations and in view of the recent construction of numerous high dams of "less than gravity" section, does it not appear that there are strong tendencies toward building future dams of other than the concrete gravity type?

EDWARD GODFREY,* M. AM. SOC. C. E. (by letter).†—The entire subject of dams, in the writer's judgment, needs to be thoroughly and freely discussed. The design of dams will never be on a sound footing until the strenuous objections to standard methods of a small, very small, but growing minority of engineers are met and clarified.

The writer is one of those who hold that standard design is woefully inadequate and has been responsible for practically all the large number of failures of masonry dams. The one great and serious fault can be simply stated: It is neglect of under-pressure. That it is in truth standard to ignore under-pressure in the design of dams can be proved in many ways. In the first place very few books on the design of dams mention or give any prominence to under-pressure. Many of those that do mention it, treat it as though it were merely a refinement—something for the designer's judgment. Some writers even refer to it as a bugaboo.

That it is standard to neglect under-pressure in designing dams is also shown by the statement of Mr. Wiley‡ that in many, if not most, straight gravity dams, no allowance has been made for uplift.

There has scarcely ever been a failure of a masonry or concrete gravity dam where the entire action of the dam cannot be fully explained by supposing that water under pressure worked its way under the dam or into a horizontal joint and lifted the dam, allowing it to float away or to be overturned; and there has never been a dam lifted and pushed down stream where the mass and base width were sufficient to hold it in place against assumed up-stream and under-pressure. Unless the truth of these statements can be denied, they stand as an unanswerable argument for unconditional provision for under-pressure and against standard methods that ignore under-pressure under any condition whatever. It is no answer to state a contrary opinion. The only refutation of these statements of any value is the citing of one or more dams the failure of which can not be explained by under-pressure, or of one which floated off, as did the St. Francis Dam, although the base width was 85% of the height instead of 65% as was that dam.

In reading Mr. Wiley's paper§ one would be led to believe that under-pressure is of little consequence and internal pressure, even in porous concrete, is non-existent. From Mr. Henny's paper‖ one would be led to believe that under-pressure is easily averted or avoided.

Mr. Wiley states‡ that:

"The absence of stress due to uplift in the body of the concrete appears from the consideration that the water can act only in the voids. The pressure

* Structural Engr., Pittsburgh, Pa.

† Received by the Secretary, December 11, 1929.

‡ *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2323.

§ *Loc. cit.*, p. 2318.

‖ *Loc. cit.*, p. 2327.

in a void in the concrete is in all directions, and considering only the vertical pressures in a single void, it might appear that a tensile stress in the concrete would result; but considering that the upward and downward forces in each void are opposed and cancelled by equal downward and upward forces in the voids immediately above and below, it is readily seen that the summation of the vertical forces in the voids is zero, and that no vertical stress in the concrete can result from water pressure in the voids. This has also been shown by the application of enormous pressures to a liquid without effect upon the concrete or other porous substance immersed in it."

This is a new line of reasoning which, if valid, calls for a recasting of the theory of pressures. It would mean that a liquid could be carried slowly through a spongy mass under high pressure without exerting tension on the pipe. It is easy to see that a spongy mass embedded in a fluid under pressure is not subject to tension in its own fibers by reason of the pressure; but, at the same time, if there is enormous pressure there is enormous tension somewhere that exactly balances it. The tension is not in the sponge but in the vessel containing and restraining the fluid. Suppose that the sponge itself is the vessel; suppose the outer skin of a sponge were elastic and impervious and that the sponge were filled with water under pressure. Every fiber of that sponge would be subject to tension. It requires no test to prove this. If there were a spongy layer in the concrete of a dam, and water entered that and was prevented from escaping on the down-stream face (and the caretaker would see to it that the joint would be sealed up), there would certainly be tension in the concrete, a tension exactly equal to the pressure. This is directly contrary to Mr. Wiley's statement of the case, but it is the law of fluid pressure and of mechanics.

Furthermore, this is just what happened in a remarkable case of the flotation of a pier* which split at a construction joint in the concrete. The upper part, weighing about 1 000 tons, was lifted by pressure of water that entered this joint in the seemingly solid concrete. The great weight of the pier and the friction against the coffer-dam, as well as what tensile strength the concrete possessed, were not sufficient to overcome the water pressure in the pores of this concrete, which, by Mr. Wiley's statement of the case, should be balanced and equal to zero.

The several means used to inhibit under-pressure in dams are: Cut-off walls, grouting fissures, and under-drainage.

Consider, first, cut-off walls. If water under-pressure has an outlet, the pressure will be reduced. If it has no outlet, the pressure will tend to reach that of the head, or the vertical distance to the free water surface. The distance through which the water and the pressure travel has influence only as affecting the time necessary to accumulate the full head. The value of cut-off walls is, therefore, simply to delay the accumulation of pressure. They do not prevent it, unless the water has a more or less free outlet.

The fact is that several dams have failed that had one or more cut-off walls, but were too light for stability. Another fact is that pressures have been measured and found to be very considerable down-stream from cut-off walls.†

* *Engineering News-Record,* March 13, 1904.

† *Transactions,* Am. Soc. C. E., Vol. 93 (1929), pp. 1532, 1537, and 1543.

If cut-off walls prevented pressure of water under a dam, there would never be a failure of a dam having a cut-off wall; because if the water were not in the foundation exerting pressure, even the standard reason for dam failure, according to all reports, would have no basis in fact.

Grouting fissures in a foundation is considered to be the one unfailing inhibitive against under-pressure. The basic idea behind this is that water can enter only where there is an actual fissure—an opening. A crack would not be classed as a rock fissure; and, furthermore, it would be quite impossible to grout up a crack. In fact, there are fissures that it would be practically impossible to fill with grout, even under pressure. Experience in grouting thin cracks has proved that the water of the grout will penetrate the crack and leave the cement. It stands to reason that solid cement or colloidal cement will not penetrate thin crevices where water may freely go. It is a well-known fact of hydraulics that water will exert its full pressure in the finest crevice.

Furthermore, it is not always possible to find all the rock seams. Some of these may be in a horizontal direction just beneath the base of a dam, and drilling would not disclose their presence. The cracks most difficult to grout are the horizontal ones, and these are the greatest menace to the stability of the dam.

In view of the facts just stated the stability of a dam that is dependent on the inhibiting of under-pressure because all rock seams are supposed to be fully grouted and impervious, is seen to be precarious.

The third method of avoiding under-pressure is drainage. This requires an expensive system of piping. There is no way to determine the effectiveness of the system. It is likely to silt up and be made useless. In cold climates the outlet is liable to freeze. The failure of more than one dam that occurred in cold weather was doubtless due to freezing of the weep-holes.

Water, working toward the drainage pipes under a dam, must of necessity exert pressure. This pressure will act against the base of the dam; hence, the only relief will be in the immediate neighborhood of the drainage pipes. Under-drainage is then, also, seen to be of precarious and unknown value.

Still another method used to prevent under-pressure is the driving of sheet-piling. This, too, has been found to be ineffectual in stopping pressure.* This method could be used only where the soil admits of driving sheet-piles, and such soil, not being solid and strong rock, would be classed by some as unsuitable for the foundation of a dam.

The evidence that under-pressure exists in many dams and may have a large value in any dam is overwhelming.† The possible pressure under the St. Francis Dam, for example, was hundreds of millions of pounds, which is a gigantic force to provide for, and a still more gigantic force to ignore.

It is pertinent to look into the arguments, the tests, and the ideas that influence some engineers to ignore under-pressure in the design of dams. The idea persists that, unless there is tail-water, there can be no flotation. This is born of the notion that, unless an object is immersed, it cannot float. Designers

* *Transactions*, Am. Soc. C. E., Vol. 93 (1929), pp. 1537 and 1556.

† *Loc. cit.*, pp. 1317 and 1527.

lose sight of the fact that it is in no sense the influence of water pressure on the side of an object that makes it float; it is solely the pressure of the water beneath it.

It is universally conceded that, where there is tail-water, there is flotation, or buoyancy equivalent to the loss of weight for the depth of that tail-water. It certainly is not pressure of water on the sides of the dam that is responsible for this buoyancy; it is, of necessity, pressure of the water beneath, and that water must work its way under the dam. There is greater facility for water and pressure to enter from the up-stream side than from the down-stream side because of the greater head.

One argument for adhering to the standard method of designing dams and ignoring under-pressure is that there are dams standing which are not proportioned for any under-pressure. The St. Francis Dam was in that class just before it failed. A large number of other dams that have failed stood in that class for as much as 40 and 50 years.* Some of these dams failed under a head of water lower than they had resisted in previous years.

A fact lost sight of is that it requires some time for water to penetrate and still more time for the pressure to build up. A high-water period may not last long enough for a destructive pressure to accumulate, and the false notion is fostered that the dam is stable. In other structures loads have an immediate effect, and a test load is conclusive.

Furthermore, time has a controlling influence because the accumulation of silt may serve to clog the outlet where water escapes below a dam and thus allow the building up of pressure. The stopping of leaks below a dam, either by Nature or by the caretaker, is equivalent to driving a gigantic wedge beneath it.

The statement is sometimes made that water beneath and in a dam is present because of capillarity and that the dam is, therefore, not subject to pressure. It is well known that water leaks out below dams. If that is an exhibition of capillarity, there is a chance for some one to invent perpetual motion. Just suck up a lot of water to a high level by a system of sponges, and let it run away as free water to run turbines, and the question of power is solved.

It is argued that pressure on the bottom of an object (in sand, for example) can be only a fraction of the computed head because there is contact on part of the surface of a grain of sand where there could be no pressure. This argument ignores the fact that the next layer of sand grains will press against the one in contact and make up fully for any such contact.

Experiments have been made tending to show greatly reduced pressure by reason of contact in sand grains.† Strangely enough, this is attributed by some to voids in the sand. Such voids are a volumetric property; pressure is purely a superficial matter. There could be no relation between sand voids and pressure of water in those voids because pressure is totally independent of volume.

* *Engineering News*, April 9 and June 4, 1903; *Engineering Record*, April 27, 1912, p. 476.

† *Transactions*, Am. Soc. C. E., Vol. 93 (1929), p. 1334.

Tests of such properties made on only a few inches of water are unreliable, as many uncertain and unknown elements enter. For example, unless the small cylinders are plunged deeply into the sand, practically full head is shown by their rising out. In the deep position, however, the starting friction (augmented by pressure on top of the sand), air bubbles in the sand, and the time element for the accumulation of pressure, combine to show an apparent upward pressure of only a fraction of the actual head.

In the well-known process of jetting piles in sand one has only to apply the water at the bottom of the pile, and it will sink in. Let the sand settle around the pile, and it is very difficult to drive or to pull it.

Air bubbles and cohesion in the sand due to the presence of clay would have a measurable influence in tests where only a few inches of head is considered. A few inches of sand could be held together by these means and the lifting action would account for part of the head.

Tests of high heads have shown, in dams and dry docks and in laboratory experiments, that under-pressure is practically 100% after sufficient time has elapsed for the pressure to build up.* In the face of these actual practical tests it is foolish to set up others on miniature models, with all the attendant uncertainties already mentioned, and to design dams on the basis of these small tests, setting aside the proofs of large-sized models and actual experience of failures.

Pressure in so-called impervious soil, while it may take more time to build up, cannot be assumed to be less than in sand. The soil above and below the tunnels in New York, being silt and clay, would seem to be classed as impervious; but this soil shows most remarkable response to fluid pressure. It is observed that the tubes sink and rise as tide rises and ebbs. This demonstrates, not only that this "impervious" soil transmits and exerts pressure, but the further fact that it requires some time for this pressure to travel through the mass; for if the pressure were transmitted at once, there would be no difference between the differential of top and bottom pressures at high and low tides, and thus there would be no movement of the tubes.

The writer believes that in practically every dam that has failed the reported reason has been "foundation failure"; but in no case, so far as he has been able to learn, have the successive steps of that foundation failure been described or analyzed. Excessive soil pressure due to weight cannot account for it, because that would merely make the dam sink. The earth cannot wash away unless there is a crevice through which water can flow; and, so long as the dam is in contact with the soil, there can be no such crevice.

The term, blow-out, is sometimes used to describe the failure. A blow-out of soil connotes high pressure, and high pressure under a dam is something non-existent, according to those who make this explanation and fail to see that that high pressure would lift and float a dam not proportioned to resist it long before it would issue in a blow-out.

It would be impossible for rock, no matter how poor a quality, to issue in a blow-out. Only fluid or semi-fluid soil could be forced out from beneath the base of a dam.

* *Transactions*, Am. Soc. C. E., Vol. 93 (1929), pp. 1330, 1331, 1333, and 1354.

It is the foregoing arguments against the neglect of under-pressure that make it incumbent on the Engineering Profession to do one of two things, namely, either to answer these arguments in a free and open discussion or to revise their standards of designing dams.

Mr. Wiley mentions* the economic advantage of arch action in dams. Reading his remarks, one cannot fail to gather the idea that merely curving the dam in plan makes an arch of it. Nothing is said of the abutment or the need of an abutment. A curved dam is merely a curved dam; it is not an arch, unless it has adequate abutments at the ends of the curves. Dams like the St. Francis Dam are not arches, for they taper down to practically nothing at the very place where gigantic abutments should be located. If suitable abutments for such long dams, not in rock gorges, were included in the design, the seeming economy would vanish. At least, two long, curved dams have failed because their designers considered them as arches, but supplied no abutments.† This is another of the major errors in standard dam design.

* *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2321.

† *Engineering News-Record*, October 13, 1926, p. 616.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

EFFECT OF TURBULENCE ON THE REGISTRATION OF CURRENT METERS

Discussion*

By Charles S. Bennett, M. Am. Soc. C. E.

Charles S. Bennett,† M. Am. Soc. C. E. (by letter).‡—The data presented in this paper are of particular interest to hydrographers because they make possible a quantitative comparison of the effects of turbulence in streams on the various types of meters. For many years there has been considerable discussion among hydraulic engineers regarding the relative merits of the different types of meters, and the data obtained in these experiments should give conclusive evidence with respect to some of the various contentions. While a few similar experiments have been made heretofore, it appears that the work done in this series more nearly approaches actual field conditions than any previous attempts.

It has long been recognized that meters of the cup type will over-register when exposed to transverse or reverse currents and that the screw-type meters tend to under-register under similar conditions. It is interesting to note that the percentage of over-registration of the Price or cup type of meters for various horizontal angles does not greatly exceed that of under-registration in the screw-type of instruments. It would appear that, disregarding all other considerations, all types of meters now in general use (with the exception of the Haskell, high-pitch wheel, and the Hoff) are subject to about the same degree of inaccuracy under these conditions, to horizontal angles of about 30 degrees.§ However, meters of the cup type tend to under-register under the influence of obliquity of flow in a vertical direction. Since turbulence causes obliquity vertically as well as horizontally, it follows that the cup meter

* This discussion (of the paper by David L. Yarnell and Floyd A. Nagler, Members, Am. Soc. C. E., published in December, 1929, *Proceedings*, but not presented at any meeting of the Society), is published in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Engr., The Miami Conservancy Dist., Dayton, Ohio.

‡ Received by the Secretary, December 19, 1929.

§ *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2633.

will probably show less actual over-registration in a turbulent stream than is indicated by the data in Fig. 15.*

Stream-flow measurements with current meters are made ordinarily from cables or bridges at selected points, and the proportion of the total cross-section in which turbulence occurs due to piers or obstructions is relatively small. When measurements are carefully made at such sections, the results should be reasonably accurate, no matter what type of standard meter is used. However, there are so many practical advantages in the improved Price meter now used by the U. S. Geological Survey, that it seems to be the most satisfactory for general purposes.

It might be suggested that, where it is necessary to make measurements in races or other streams in a turbulent condition, readings be obtained with a cup meter as well as with a screw meter, such as the small Ott. Comparisons of the readings should give an idea of the approximate compensation to allow for inaccuracy due to angular approach of the stream filaments. It might also be well to make longer observations than usual when turbulent conditions exist; that is, make each reading of velocity cover a period of about 180 sec. instead of the customary 60 to 70 sec.

The data in Tables 1† and 2‡ result from experiments made at velocities of 2 ft. per sec. and less. From Fig. 14§ it appears that an increase of velocity up to 5 ft. per sec. makes a considerable difference in the percentage of variation for two types of meters at various horizontal angles. This comparison raises the question of the effect of higher velocities, such as occur in ordinary streams in flood, on the variation. It is hoped that further studies may be made, using experiments at velocities somewhat higher than those of this series.

The writer believes that these experiments bear out the general opinion of hydrographers that current meters cannot be expected to register correctly in turbulent water; but it should be remembered that the greatest practical value of the current meter lies in its use for the measurement of the flow of average streams, in which it is usually possible to select a measuring section where there is a minimum of disturbing conditions.

* *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2633.
† *Loc. cit.*, p. 2624.
‡ *Loc. cit.*, p. 2625.
§ *Loc. cit.*, p. 2632.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

SAMUEL GIVENS BENNETT, M. Am. Soc. C. E.*

DIED APRIL 4, 1929.

Samuel Givens Bennett, the son of John W. and Agnes (Givens) Bennett, was born at Taylorsville, Ky., on July 12, 1863. He was educated in the public schools at Owensboro and Hartford, Ky., and later studied for two and one-half years at Georgetown College, Georgetown, Ky.

In 1886, Mr. Bennett went to California where he commenced his life work, the practice of land surveying and civil engineering. His first employment was with the late E. T. Wright, M. Am. Soc C. E., of Los Angeles, Calif., in the capacity of Chainman, Rodman, and Draftsman.

In subsequent years he was engaged in the following positions: From September, 1889, to September, 1890, as Draftsman and Instrumentman with L. Friel, Civil Engineer; from September, 1890, to September, 1891, as Assistant Engineer with the Redondo Railway Company; from September, 1891, to October, 1893, as Draftsman with the Pacific Coast Abstract Company, of Los Angeles, Calif.; from October, 1893, to December, 1896, in conducting general engineering business in Los Angeles and vicinity, in partnership with Frank H. Olmsted, M. Am. Soc. C. E., and the late Burr Bassell, M. Am. Soc. C. E.; from May, 1897, to April, 1898, as Engineer with the Chicala Water Company, of Rialto, Calif.; from April to October, 1898, as Superintendent of the construction of a pumping plant and a riveted steel pipe line for the Azusa Glendora Water Company; from October, 1898, to April, 1899, in association with J. B. Lippincott, M. Am. Soc. C. E., in connection with the Pasadena, Calif., Water Supply System; from April, 1899, to January, 1903 as Assistant Resident Hydrographer for the United States Geological Survey in California; from January, 1903, to June, 1906, as Engineer in the United States Reclamation Service, having charge of the Sacramento Valley Project for almost two years of this period; in 1907, as Locating Engineer for the Los Angeles and Owens Valley Railroad Company; from 1908 to 1909, in private practice; and during 1910, as Engineer for the United Sugar Company in the State of Sinaloa, Mexico. In 1911, Mr. Bennett again returned to private practice, but was soon appointed City Engineer of Oxnard, Calif., which position he held until his death.

On March 22, 1899, Mr. Bennett was married at Redlands, Calif., to Mable J. Randall, of Minneapolis, Minn., who, with their four children, Harman R., S. Gerald, Mable Rose, and John Newton, survives him. Mr. Bennett was a member of the Baptist Church, and was greatly beloved among a wide circle of friends.

* Memoir prepared by Frank H. Olmsted, M. Am. Soc. C. E.

Of a retiring, modest disposition, he was a great student along scientific and religious lines. His hydrographic work in the High Sierras in California for the U. S. Geological Survey brought him conspicuous success. As City Engineer of Oxnard, "Sam" Bennett enjoyed the esteem and respect of the Trustees of that city during the long period of years in which he held that office.

Mr. Bennett was elected a Member of the American Society of Civil Engineers on October 3, 1906.

WILLIAM LEWIS BRECKINRIDGE, M. Am. Soc. C. E.*

DIED JULY 11, 1929.

William Lewis Breckinridge, the son of Marcus Prevost Breckinridge, M. D., and Lucy (Long) Breckinridge, was born at Louisville, Ky., on June 29, 1857.

He was a direct descendant on his paternal side of a family noted for its patriotism, oratory, and statesmanship. After first settling in Virginia in 1730, John Breckinridge, his great-grandfather, migrated to Kentucky in 1780. One of his grandsons, John C. Breckinridge, became famous in political circles, being Vice-President of the United States from 1856 to 1860, and, later, running for President of the United States in opposition to Abraham Lincoln. He afterward became a General in the Confederacy and Secretary of War under Jefferson Davis.

His grandfather, the Reverend William Lewis Breckinridge II, was a Presbyterian Minister, and, at one time, served as Moderator of the combined Presbyterian Churches of the United States. While many of the Breckinridge family espoused the Southern cause, Marcus Prevost Breckinridge and his descendants allied themselves with the Union in the controversies leading up to the Civil War.

On his maternal side, the Long family was quite as prominent; his grandfather, Stephen Harriman Long, was one of the foremost engineers of his day. From 1818 to 1826, he supervised explorations between the Mississippi River and the Rocky Mountains, Long's Peak, the highest summit in those mountains, being named after him. In 1829, he published a "Railroad Manual", the first book of its kind in the United States. From 1827 to 1830, he was Chief Engineer of Surveys, in connection with the building of the Baltimore and Ohio Railroad; later, he became Engineer in Chief of the Western and Atlantic Railroad, in Georgia, during which time he introduced a system of curves on location work. He served for a time on the Board for Improvements of the Mississippi River, and was made Colonel of Engineers in 1861, shortly before his retirement in 1863.

William Lewis Breckinridge spent his boyhood days in Alton, Ill., attending the public schools. After graduating from High School, he went to the

* Memoir prepared by a Joint Committee of the Society, the Western Society of Engineers, and the Chicago Engineers Club, consisting of A. W. Newton and Elmer T. Howson, Members, Am. Soc. C. E., and Charles W. Melcher, M. West. Soc. E.

Worcester Military Academy, Worcester, Mass., and from there to Washington University, St. Louis, Mo., from which he was graduated with the degree of Civil Engineer in 1879. In June, 1907, Washington University conferred upon him the Honorary Degree of Master of Arts.

He entered the service of the Chicago, Burlington, and Quincy Railroad Company on July 15, 1879. Beginning as an Assistant Engineer, he was subsequently made a Division Engineer in 1880; Engineer of Iowa Lines in 1890; Assistant Superintendent of Iowa Lines in 1898; Chief Engineer in 1899; Chief Engineer of Maintenance in 1904; and Chief Engineer of the Chicago, Burlington, and Quincy System, then under Federal Control, in 1918. He continued in this capacity until the end of Federal Control in 1920, when he was appointed Assistant Chief Engineer of the System, which position he held until his retirement in April, 1923.

In his service with the Burlington Railroad, extending over a period of nearly half a century, Mr. Breckinridge saw the Lines East of the Missouri River grow by construction and by acquisition from a group of loosely knit light traffic lines, comprising a total of 1 857 miles in 1880, to a highly developed system of 4 410 miles in 1923. This period was one of intensive, as well as extensive, development, much of which was under Mr. Breckinridge's supervision. He was in charge of the re-alignment and revision of grades on the Burlington's main line across Iowa from 1898 to 1902, which comprised some of the heaviest work of its kind up to that time.

Under his administration, the Burlington extended its line into the Southern Illinois coal fields, and then rebuilt existing lines from Centralia, Ill., north to St. Paul, Minn., providing a maximum grade of 0.3% for this distance of more than 600 miles. Coincident with this program, numerous large terminals were built, including the gravity classification yards at Galesburg and Hawthorne, Ill., and the new locomotive shops at West Burlington. Particularly outstanding for its day was the track-elevation work between Chicago and Clyde, Ill., one of the earliest projects of this magnitude to be undertaken.

Mr. Breckinridge was a man of unusual personality, possessing a dignity of bearing, yet with a friendliness that inspired respect and admiration. He was a gentleman in every sense of the word, considerate of others to an unusual degree—a trait which won for him the unreserved loyalty and friendship of all those with whom he came in contact. Naturally of a retiring disposition, his ability and personality were known only to those with whom he was associated, but by them he was held in the highest esteem.

On October 15, 1891, he was married to Irene Waples, of Alton, Ill., who, with two sons, William Lewis and Frank Prevost, survives him. His religious affiliations were with the Protestant Episcopal Church.

He became a member of the Western Society of Engineers and of the American Railway Engineering Association in 1899, and of the Chicago Engineers' Club in 1906.

Mr. Breckinridge was elected a Member of the American Society of Civil Engineers on October 7, 1903.

GEORGE MARTIN ALOYSIUS ILG, M. Am. Soc. C. E.*

Died June 28, 1929.

George Martin Aloysius Ilg was born in Chicago, Ill., on November 26, 1881, the son of John J. and Christine (Schnebelen) Ilg, being one of ten children. Mr. Ilg received his early education in the public schools of Chicago and was graduated from Calumet High School in 1901.

His practical experience in engineering began shortly after his graduation, when he went to work for the American Bridge Company. He served four years with this Company as Shop Inspector, Tracer, Detailer, and Checker on steel work for bridges and buildings.

In 1905 he entered the School of Civil Engineering at the University of Illinois, paying his own way with funds he had saved during the previous four years and with the help of earnings as a Draftsman for the Kenwood Bridge Company during the summer vacations. He completed the full four-year course and was graduated in June, 1909, receiving the degree of Bachelor of Science in Civil Engineering.

After his graduation from college Mr. Ilg began his engineering work with the Chicago, Milwaukee and St. Paul Railroad Company and was soon given charge of the design of concrete and steel bridges and other structures. In 1911, he entered the employ of Holabird and Roche, Architects, as a Designer in the Engineering Department. During the next seven years, he was responsible for a large part of the firm's design work in both reinforced concrete and structural steel buildings.

In 1916, in connection with his regular work, he had occasion to design some long-span steel arches for an armory. He became interested in arch design to the extent of devoting considerable additional study in his spare time to this subject, which resulted in his preparation of a thesis on arches. Based on this, together with other requirements, Mr. Ilg received the degree of Civil Engineer from the University of Illinois in June, 1916.

In May, 1917, he left the employ of Holabird and Roche and entered the First Officers' Training Camp, at Fort Sheridan, Ill. He held a commission in the Reserves as Captain of Engineers, which commission was confirmed after training service at Fort Sheridan, Ill., and at Fort Leavenworth, Kans. At the completion of his training he was put temporarily on the inactive list and, while waiting for a call, entered the employ of the Stone and Webster Corporation, at Philadelphia, Pa., on the structural design of buildings for Hog Island Shipyard. His work here apparently was deemed as important as the more active duty in France, as he was not called for active service, but remained on the Hog Island work for the duration of the World War.

At the end of the war, in the fall of 1918, Mr. Ilg returned to Chicago and became Structural Engineer for Jarvis Hunt, Architect, in charge of design and construction. After two years in this position, he resigned on account of

* Memoir prepared by F. E. Brown, M. Am. Soc. C. E.

poor health resulting, it was thought, from overwork during the war. His health did not improve, and he suffered a severe nervous breakdown. Seeking outside work, he moved to Pierre, S. Dak., in 1920, where he accepted a position as Assistant Bridge Engineer for the South Dakota Highway Commission.

After two years on this work, Mr. Ilg's health was very much improved, and in 1922 he moved to St. Paul, Minn. Here, he served for some time in the City Engineering Department, where he was engaged on the design of a number of bridges, including the Ford Bridge between Minneapolis and St. Paul. Later, he entered private practice as a Civil and Structural Engineer, with headquarters in St. Paul.

In January, 1926, Mr. Ilg returned to Chicago and entered the employ of Smith and Brown, Engineers, Incorporated. He was with this firm until his death.

He will long be remembered and esteemed by his associates for his kindly and genial disposition, his friendliness to others, and his interest in the success of his fellow workers. He enjoyed his profession fully and always took a keen delight in any unusual problem, no matter how difficult or tedious. His singularly alert mind and his fine training and experience made his services especially valuable to his associates.

On June 27, 1910, Mr. Ilg was married to Gretchen Zeit, the daughter of Dr. F. Robert Zeit, of the Medical Faculty of Northwestern University. He is survived by his wife and an only daughter, Marguerite.

He was a Mason, a member of the Triangle Engineering Fraternity, Hiram Sleifer Post of the American Legion, the Forty and Eight Club, and the Western Society of Engineers. He was also a Captain, Engineers Reserve Corps, U. S. Army.

Mr. Ilg was elected a Member of the American Society of Civil Engineers on March 12, 1923.

CHARLES NEIL MC DONALD, M. Am. Soc. C. E.*

DIED JULY 5, 1929.

Charles Neil McDonald, the son of Alexander and Johanna McDonald, was born on August 25, 1862, near Sidney, Nova Scotia. As a boy he attended the grade schools of Sidney, and there, too, he served an apprenticeship as a carpenter.

In 1882, after living for a short time in Boston, Mass., Mr. McDonald moved to the Dakotas, where he was employed as Bridge Carpenter and Bridge Foreman by the St. Paul, Minneapolis, and Manitoba Railway Company (now a part of the Great Northern Railway Company).

In 1884, Mr. McDonald moved to the Far West and located at Celilo, Ore., where he entered the employ of the Oregon Railroad and Navigation Company as Bridge Foreman on the construction of stone, concrete, iron, and timber bridges.

* Memoir prepared by S. Murray, M. Am. Soc. C. E.

In 1890, he was appointed Superintendent of Construction for Robert Wakefield and Company, Contractors, of Portland, Ore., and for eighteen years he continued in active charge of all the numerous large construction projects undertaken by that firm. During this period, Mr. McDonald's work covered a wide scope and included docks and harbor improvements at various points along the Pacific Coast, numerous bridges for the Union Pacific System, all the bridges on the Astoria and Columbia River Railway, and several large structures for the City of Portland. He was also retained from time to time to represent the Chief Engineer of the Oregon Railroad and Navigation Company in the execution of difficult bridge foundations and in the solution of various erection problems.

In 1908, his activities carried him into the Far North. He associated himself with the Copper River and Northwestern Railway Company which was being extended into the interior of Alaska, and was placed in charge of all the bridge construction on that line.

This work, which included the famous Miles Glacier Bridge and the Kuskolina Cantilever, was crowned with notable success in the face of most adverse circumstances. Operations were carried on far from a base of supplies; the season of the year in which climatic conditions were favorable for construction was limited to only a few months; and the work had to go on during the long stormy winters of the Copper River country and in temperatures almost unbelievably low.

Large masses of ice are discharged from Miles and Child's Glaciers into the Copper River during the summer months, and for that reason the Miles Glacier Bridge which crosses that stream was originally designed for cantilever erection. To advance the opening of the railroad a full year, however, the plan was changed, and it was decided to carry on substructure work during the winters and, at the same time, to erect the spans of falsework supported on ice. The plan was a success. The ice moved out in an unusually early spring thaw just twenty minutes after the bridge was self-supporting. This dramatic incident is well described in Rex Beach's novel, "The Iron Trail."

In 1912, Mr. McDonald returned to Portland and again associated himself with Robert Wakefield and Company. For several years he was the active head of that firm in charge of the execution of important river and harbor work and large bridge projects, involving the expenditure of many millions of dollars. Included in this work was the erection of the Harriman Bridge (double-deck railroad and highway lift) across the Willamette River, at Portland, and the Interstate Bridge across the Columbia River, at Vancouver, Wash.

In 1920, he became Vice-President of the Gilpin Construction Company of Portland, and, from that time until his death, he directed its more important operations, among which were bridges across the Columbia, Willamette, and other large rivers of the Pacific Northwest.

Mr. McDonald possessed to a rare degree the good judgment that is acquired only in the stern school of experience, but he combined with it a technical knowledge and designing ability worthy of a man of great formal training. His counsel was sought by many, especially by the younger men, whom he seemed to

understand. His life was marked by a succession of great achievements, bold and daring in conception, taxing amazing resources of skill and energy in execution, yet uniformly successful in realization.

Deeply religious, and of a modest and retiring disposition, generous, honorable, and upright in all things, he was an inspiration to those who came in contact with him. No man was loved more by his associates. His passing has left an empty place which they feel can never quite be filled.

In 1890, he was married to Carrie Harrison, who survives him, together with two daughters, Mrs. Joan Atwater, of Eugene, Ore., and Mrs. Jessie Acklen, of Raymond, Wash.

He was a member of various Masonic bodies and other fraternal orders, and was affiliated with the Methodist Episcopal Church.

Mr. McDonald was elected a Member of the American Society of Civil Engineers on January 17, 1927.

ALEXANDER BAIN MONCRIEFF, M. Am. Soc. C. E.*

Died April 11, 1928.

Alexander Bain Moncrieff was born in Dublin, Ireland, on May 22, 1845, a descendant of an old Scottish family of Perth which numbers among its members a noted lawyer, a famous divine, and many others distinguished in military and civil affairs.

Mr. Moncrieff spent several years at the Belfast Academy, after which he was articled to the late Mr. E. Miller, Chief Engineer of the Great Southwestern Railways in Ireland. He served his apprenticeship in the railway shops at Inchicore, Dublin, and had seven years' experience in the workshops and in the office of the same Company. This period included twelve months spent in the blacksmith's shop, where he worked from 6:00 A. M. until 5:00 P. M., in close contact with a side of life that developed in him a feeling of keen sympathy for his fellow workers, to whom he always said he owed a great deal.

At the close of this training and experience Mr. Moncrieff went to Glasgow, Scotland, as Draftsman for the Dubbs Works, at Govan, where he was engaged mainly on the design and construction of locomotives. Because of business changes, he returned to Ireland and took charge of the rebuilding of a large water-driven flour-mill and malthouse at Milford-on-Barrow. On the completion of this work, Mr. Moncrieff assumed charge of a small workshop in London, England. After a short interval, however, he answered an advertisement issued by the South Australian Government for an efficient Draftsman. His application met with immediate success and he accordingly took ship to Australia, where he arrived in February, 1875. As Engineering Draftsman he was chosen to assist in the design of fortifications and in this capacity he was obliged to travel to Melbourne, Sydney, and Brisbane, Australia.

In 1879, Mr. Moncrieff was appointed Resident Engineer of the South Australian Railways, taking charge of the line as it was gradually extended from Port Augusta to Oodnadatta. In 1888, on the retirement of the late

* Memoir prepared from information furnished by Alex. S. Moncrieff, Esq., Adelaide, South Australia.

H. C. Mais, M. Am. Soc. C. E., he was promoted to the office of Engineer-in-Chief, which position he held for twenty years. In addition to his duties in the Railways Department, he had charge of the construction and maintenance of harbors, jetties, and lighthouses, as well as the construction of water-works and the conservation of water. The construction of the Happy Valley Water Works and of the Outer Harbor, Adelaide, are two outstanding monuments to his ability, determination, and zeal—true evidences of engineering achievement of which any engineer might justly be proud. They stand as a silent memorial to a career of exceptional usefulness and conscientious public service.

On the retirement of the late Mr. Alan G. Pendleton, Mr. Moncrieff succeeded to the office of Railways Commissioner. As such, he served under nineteen Commissioners of Public Works, from all of whom he received hearty support in his work. Under Mr. Moncrieff's supervision many improvements were accomplished, including the completion and putting into service of the Mile-End Goods Yard, by which means up-to-date freight accommodation was provided for the City of Adelaide and its suburbs; new siding accommodations at Port Adelaide; the installation of appliances for the prevention of fire at railway stations; the improvement of sanitary arrangements at stations and of the facilities on the Woodville and Henley Beach Lines; the increase in and the improvement of the rolling stock of the railways; the fitting of the Westinghouse brake to a large number of freight trucks, which made possible the delivery of parcels in the suburban area by motor vehicles; the establishment of an inquiry office at the Adelaide Railway Station (an opportunity was also given the apprentices to improve themselves by attending classes at the School of Mines); the placing of railway fares on a scientific basis; the teaching of first aid to a large number of the staff and the planting of gardens at stations which was encouraged wherever possible. Mr. Moncrieff also was the "father" of the South-Eastern Drainage Project in which he took great interest and which at its inception he personally explained to conferences of landholders held at Narracoorte and Millicent, Australia, ensuring their hearty support of his proposals.

In 1916, after forty-two years of service in several capacities, Mr. Moncrieff retired as Railways Commissioner. During this entire period his only vacations were those spent in traveling on Departmental business. When asked once by a representative why he did not take a holiday he replied, "I do not feel that I need a holiday and the fact that I have never been ill proves it." Mr. Moncrieff was a living justification of the gospel of hard work.

He was the first Chairman of the Municipal Tramways Trust and held this position from its inception in 1907 until January, 1922. He was a member of the Institution of Civil Engineers of Great Britain, and in December, 1909, under the hands of the King of England, was made a Commander of the Order of St. Michael and St. George.

Until shortly before his death, Mr. Moncrieff had enjoyed the most robust health, taking a constitutional walk to and from the city several times a week. Also, before his retirement, he walked daily to his office from his home. He was a lay reader of the Church of England, and was greatly interested in

church work. Gardening, reading, mechanics, and Free Masonry were also among his recreational pursuits and in these releases from his strenuous official career he found joy and relaxation.

Mr. Moncrieff was a man of decided opinions and of great personal force. The Secretary to the Railways Commissioner, Mr. C. J. Boykett, in referring to Mr. Moncrieff, has stated that officers who were intimately associated with him in practically all the positions he occupied testify to his outstanding capability, versatility, and energy displayed in carrying out his undertakings. He was held in high esteem for his gentlemanly and honorable conduct by all those with whom he was associated in the railway service, and was regarded with warm affection by those with whom he was more intimately acquainted. A true gentleman, kindly and most gracious by nature, his passing is mourned by a great company of associates and friends. He is survived by his widow, Mary Bronson Moncrieff, a son, and a daughter.

Mr. Moncrieff was elected a Member of the American Society of Civil Engineers on July 4, 1894.

HARRY FAY ROACH, M. Am. Soc. C. E.*

Died July 28, 1929.

Harry Fay Roach was born in St. Louis, Mo., on May 7, 1871, the son of Harry E. and Sarah (Haley) Roach. His mother was a native of Kalamazoo, Mich. His father was an Architect and Builder and the son grew up in an atmosphere of construction, spending his spare time in his father's office and on work in the field. He thus acquired considerable valuable training in his early youth.

Mr. Roach was graduated from the St. Louis Manual Training School in 1889, and in the fall of that year entered the Massachusetts Institute of Technology. He remained there until the summer of 1891, taking the course in Architecture, after which he entered into partnership with his father for the practice of his profession in St. Louis. In 1894, he traveled through England, France, Belgium, Germany, Switzerland, Italy, and Greece, studying the architecture of those countries. His partnership with his father was terminated in 1903, and thereafter, until the time of his death, he was engaged in architectural and engineering work. He designed many important buildings in St. Louis, among which are the Hamilton-Brown Building, the Times Building, the Buckingham Club, and the Syndicate Trust Building.

Mr. Roach was a mathematician of note and had an ingenious mind and mechanical ability of high order. He developed the means of generating electricity for the lighting of railway trains as early as 1899. He patented railway joints and angle-bars, designed rail sections, and invented a method of photographing the deflections in railway track and structures under moving loads by the use of motion picture camera and reflectors. All the work of invention and research that Mr. Roach did, was actuated by the love of scien-

*Memoir prepared by a Committee of the St. Louis Section consisting of F. G. Jonah, C. D. Purdon, and Baxter L. Brown, Members, Am. Soc. C. E.

tific investigation and not with the hope of any pecuniary reward. His discussion of engineering subjects was always instructive, and his friendship was prized by a large circle of professional men in St. Louis.

Mr. Roach was affiliated with the following organizations: The Third Baptist Church, of St. Louis; the Masonic Order; the American Institute of Architects; the Society of American Military Engineers; and the St. Louis Railway Club. After a severe illness of three months, he passed away on July 28, 1929.

In 1893, he was married to Mary E. Gallop, who survives him. He leaves also two daughters, Mrs. Donald B. Pheley, of Pasadena, Calif., and Dorothy E. Roach, of St. Louis; and four sons, Harry F. Roach, Jr., and Alan Douglas Roach, of St. Louis, Alden G. Roach, Assoc. M. Am. Soc. C. E., of Altadena, Calif., and Noyes H. Roach, of Joliet, Ill.

Mr. Roach was elected a Member of the American Society of Civil Engineers on January 19, 1925.

EDWARD AHLERT STUHRMAN, M. Am. Soc. C. E.*

Died May 31, 1929.

Edward Ahlert Stuhrman was born in New York, N. Y., on December 18, 1884, where his early life was spent and where he prepared himself for college.

He was graduated from Columbia University in the Class of 1907, with the degree of Bachelor of Science in Civil Engineering. Later, he was granted a license by the State of Illinois to engage in the practice of Architecture and was a Registered Engineer in the State of Florida.

After his graduation, Mr. Stuhrman was employed for a short time as Timekeeper and Foreman, by the Carlin Construction Company, of Brooklyn, N. Y. He then entered the service of the Turner Construction Company of New York, as Draftsman and Assistant Engineer, resigning to become connected with the Industrial Engineering Company of New York.

Upon leaving New York, he went to Chicago, Ill., where he was employed by the Chicago, Milwaukee and St. Paul Railway Company, serving with that Company for several years. From Chicago, Mr. Stuhrman went to Baltimore, Md., with the Arthur Tufts Company, and was in charge of the design and construction of warehouses and concrete water tanks for the Coco Cola Company at Baltimore, Los Angeles, Calif., New Orleans, La., Kansas City, Mo., Winnipeg, Man., Canada, and Atlanta, Ga. While in Atlanta, he was engaged as Engineer on the Candler Warehouse, Emory University Buildings, and Wesley Memorial Hospital.

Mr. Stuhrman moved to Miami, Fla., in 1924 and during the few years of his residence there, he was connected with some of the largest construction projects in Southern Florida. Some of the more important buildings on which

* Memoir prepared by Meldrim Thomson, Esq., Miami, Fla.

he served as Engineer are the Congress Building, Seybold Building, First Trust and Savings Bank Building, City National Bank Building, Miami Senior High School, Gulf Stream Apartments, at Miami Beach, Whitehall Hotel, Palm Beach, Rolyat Hotel, at St. Petersburg, and the passenger station and hangars for the Pan-American Railways at Miami.

Mr. Stuhrman was a member of St. George's Protestant Episcopal Church, of New York. He was a Thirty-Second Degree Mason and a member of Yaraab Temple, A. A. O. N. M. S., of Atlanta, Ga. At the time of his death, he was President of the Miami Engineers' Club. He was also a member of the American Concrete Institute, and the Rotary Club.

In Mr. Stuhrman's untimely death, Florida has lost one of her most brilliant adopted sons.

He was married in Brooklyn, N. Y., on August 26, 1908, and is survived by his widow, Violet L. Stuhrman, and two sons, Everard and Ahlert.

Mr. Stuhrman was elected a Member of the American Society of Civil Engineers on September 10, 1918.

ALEXANDER MILLER TODD, M. Am. Soc. C. E.*

Died September 11, 1929.

Alexander Miller Todd, the son of George T. and Marion (Miller) Todd, was born September 28, 1874, in Rankin County, Mississippi. His early education was obtained in the public schools of Rankin County. He matriculated at Texas Agricultural and Mechanical College in September, 1890, and was graduated in June, 1894, with the degree of Bachelor of Civil Engineering.

On the completion of his college course, Mr. Todd entered the service of the Mississippi River Commission, Third District. He started as a Rodman at the bottom of the Engineering Department, and by hard work advanced rapidly through the grades of Levelman, Transitman, and Inspector, to become a Superintendent of Construction in 1900.

His early survey work brought him into intimate field contact with the Mississippi River and gave him a practical background that proved invaluable later when he reached a position to study the problems of river control. From the first, he manifested a lively interest in the behavior of the Mississippi, and his keenly analytical mind early began to consider the difficult problems of controlling the mighty "Father of Waters".

Mr. Todd devoted his whole professional life to the Federal Service. From a Superintendent of Construction in 1900, he rose steadily through the various grades in the Engineer Department, until at the time of his death he held the position of Senior Engineer. His entire service was in what is now called the Vicksburg Engineer District.

During the war-time emergency, from 1917 to 1919, he served as District Engineer of the Third Mississippi River Commission District and directed,

* Memoir prepared by Elliott J. Tucker and George R. Clemens, Associate Members, Am. Soc. C. E.

during this trying time, the prosecution of many important works. His services were so highly regarded by the Federal Government that he was made a Special Disbursing Agent, United States Engineer Department, U. S. Army.

Mr. Todd was a practical student of many of the important flood-control problems on the Mississippi River. His studies of gauge relations were most exhaustive and were used extensively in determining levee grade lines and in predicting flood crests. He gave much time to the study of bank protection works, and his influence was felt, not only in the theoretical analysis of this problem, but in the practical design of machinery for placing revetment. In levee building, also, he became an authority. In the District Office his opinion, as to the relative merits of revetment or levee construction in the solution of a particular local problem, was the deciding factor in making the recommendations for construction.

In 1913, when it became apparent that a much extended program for Mississippi flood control would be necessary, Mr. Todd foresaw that the available supply of material for the older types of bank protection works or revetment would be insufficient to meet the demands. The use of concrete was coming into prominence, and it was soon realized that this offered a possible solution of the problem. When the articulated concrete mat was developed, Mr. Todd designed an all-steel floating plant for the manufacture and placing of this mat, which proved most successful. This plant was unique, in that it was equipped with the first known concrete mixer utilizing a horizontal distributing boom.

The demands of the 1914 Mississippi River Commission program for increased height and section of levees were such that it was apparent to the levee student that improved methods of construction would be necessary. Accordingly, Mr. Todd devoted much of his time and energy to the study of more economical methods for placing earthwork. Early experiments, with a taut-line cableway machine, were not as satisfactory as desired. Further experiments with a slack-line drag-bucket cableway were more successful and, in 1916, in collaboration with engineers of the Bucyrus Company, he designed the tower levee machine embodying this principle. This machine proved very successful and has been used with but very little change in the original design to the present time (1929). The capacity of these machines is approximately 150 000 cu. yd. of earthwork per month—something impossible with the older type of equipment.

In addition to his work on the main Mississippi River, Mr. Todd directed certain surveys on the tributaries. In 1910, he made the field survey and report for the power development on the Upper Ouachita River, near Hot Springs, Ark., out of which has developed the project (Federal Power Commission No. 271) for three power dams—Remmel (built), Carpenter (under construction), and Blakely Mountain (proposed).

Personally, Mr. Todd was held in the highest esteem. At more than one heated conference his was the quiet head that calmed the discussion and brought order and progress out of argument. His ability to serve in difficult capacities was attested by the fact that he was appointed in May, 1925,

by the President of the Mississippi River Commission to serve as a member of the Investigating Board of the *"Norman"* steamship disaster at Memphis, Tenn.

He was keenly conscious of his civic duties and served in many capacities in a number of organizations. He was a leader in community betterment and devoted much of his time to these interests.

Death came to Mr. Todd very suddenly, while on an inspection of the Carpenter Dam construction near Hot Springs, Ark. His life was cut short by apoplexy at the crest of his active career. Funeral services were held at the First Baptist Church, Vicksburg, with interment at the City Cemetery. He is survived by his wife, Sue C. (Carleton) Todd, to whom he was married May 17, 1898, in Chicot County, Arkansas, and by three children—Carleton R. (Lieutenant, U. S. Navy), Marion E. (Todd) Rogers, and Elizabeth A. (Todd) Burnett.

Fitting testimonial to Mr. Todd's many sterling qualities was given in the following public announcement of his death, made by the District Engineer, Major John C. H. Lee, Corps of Engineers, U. S. A., M. Am. Soc. C. E.:

"With deepest regret the District Engineer announces the death of Sr. Engineer Alexander Miller Todd, resulting from a stroke of apoplexy while inspecting the construction work at Carpenter Dam, Ouachita River, Arkansas, about 11 A. M., September 11, 1929.

"Mr. Todd died on duty, ending a career of outstanding merit and remarkable service to the United States as well as to the people of the valley. In this career of devoted and unsurpassed service, Mr. Todd identified himself with all phases of Mississippi flood control. He has not only been an invaluable student of great flood control problems, but he has exemplified a practical ability to execute work and hold the devoted loyalty of his fellow employees.

"Mr. Todd was one of the senior members of the American Society of Civil Engineers in Mississippi; an active member of the Vicksburg Chamber of Commerce and the Vicksburg Rotary Club. He was a Deacon of the local Baptist Church, a Director of the Y. M. C. A., and has held the honored respect of the entire community. To all who have known him, Sr. Engineer Todd's life has been an inspiration. Revered, admired, beloved by his fellow men, this able public servant and loyal citizen of a bereaved community will live always in the memory of his friends."

Mr. Todd was elected a Junior of the American Society of Civil Engineers on October 1, 1895, an Associate Member on October 2, 1901, and a Member on June 6, 1905.

IRVING WORTHINGTON, M. Am. Soc. C. E.*

DIED MAY 27, 1928.

Irving Worthington was born at Sauk Center, Minn., on June 19, 1868. His mother was Sarah Lewis, of New York State, and his father, Leslie Worthington, was descended from a Connecticut family which claimed a Revolutionary War ancestry on his mother's side. He was related to the late Henry Worthington, M. Am. Soc. C. E., of New York, N. Y.

Irving Worthington's education was begun in the schools of Sauk Center. Later, when the family moved to Barrie, N. Dak., he finished his common school

* Memoir prepared by William J. Roberts, M. Am. Soc. C. E.

education there and also taught for one year. He moved to Spokane, Wash., in 1889, and was enrolled in the pre-engineering course at Spokane College, where he was a student for two years.

Mr. Worthington's first engineering work in 1891 was on Government surveys under Mr. J. K. Ashley, of Spokane. From 1893 to 1903, he was engaged in the private practice of general engineering and maintained an office in Spokane. During this period, he surveyed much of Chelan and Okanogan Counties, Washington and, as Deputy Mineral Surveyor, the mineral belts and mines of Washington, Oregon, Idaho, and Montana.

In 1903, Mr. Worthington began his long experience in the irrigation field under Ernest McCulloh, M. Am. Soc. C. E., for the Yakima Development Company, of Yakima, Wash. He was active in irrigation investigations for this Company for three years. From 1905 to 1908 he was in charge of important irrigation developments for the Oregon Land and Water Company. During the next five years he was Chief Engineer of the Rogue River Valley Canal Company, Medford, Ore. This project embraced more than 50 000 acres, and included two storage reservoirs, and many miles of canal and laterals, as well as the problems of land settlement. The writer had knowledge of Mr. Worthington's work on this project from personal observation and the results of his labor are now apparent in the productiveness under irrigation in Rogue River Valley.

In 1913, he was called to be Assistant Engineer and Water Superintendent for the Fresno, Calif., Canal and Irrigation Company and continued as such until his appointment as Engineer Appraiser for the Federal Land Bank, at Spokane, which post he held until his death on May 27, 1928.

Mr. Worthington was married to Frances C. Brattain, of Spokane, in March, 1900. Mrs. Worthington, with their three children, Patricia, Hugh, and Jean, survives him.

He was a likeable man, a conscientious and painstaking engineer. Whatever work he undertook, was well done.

Mr. Worthington was elected a Member of the American Society of Civil Engineers on June 24, 1914.

ERNEST ARDEN BRUCE, Assoc. M. Am. Soc. C. E.*

DIED MARCH 8, 1928.

Ernest Arden Bruce, the son of J. A. and E. Lee (Wiseman) Bruce, was born at Ingleside, W. Va., on January 28, 1887. He spent his early life in Bluefield, W. Va., where he fitted himself for college. He entered Concord College in 1904; later, he became a student in West Virginia University from which institution he was graduated in 1908 with the degree of Bachelor of Science in Civil Engineering.

Mr. Bruce began his professional career in Bluefield, where he served as Transitman, and, later, as City Engineer. He also served in the capacity of City Engineer for the Cities of Princeton and Beckley, W. Va. From 1915

* Memoir prepared by Philip Joseph Walsh, Assoc. M. Am. Soc. C. E.

to 1917 he was engaged as Assistant City Engineer of Charleston, W. Va., and from 1919 to 1923, he was City Engineer of Charleston, after which he resumed his private practice as Civil and Sanitary Engineer.

He organized the Bruce Construction Company, which Company became extensively engaged in the general contracting business of building roads, streets, and sewers. Mr. Bruce was a member of this Company at the time of his death.

He was a member of the American Association of Engineers, and President of the State Board of Registration of Engineers of West Virginia. He was an engineer of unusual professional attainments, ever active in promoting the increased recognition of Registered Professional Engineers by the public.

Mr. Bruce ably served on many engineering committees appointed to co-operate with various civic bodies in the handling of local public problems. He was considered one of the outstanding leaders in his profession, whose judgment was sound on technical matters.

He possessed a lovable disposition which endeared him to all who were privileged to know him; his untimely passing spread universal sorrow among his friends, and was a distinct loss to his profession.

Mr. Bruce was elected an Associate Member of the American Society of Civil Engineers on April 14, 1919.

GARRETT ALEXANDER FRASER, Assoc. M. Am. Soc. C. E.*

Died April 10, 1929.

Garrett Alexander Fraser was born on February 24, 1895, in Spokane, Wash.; his boyhood home, however, was in Butte, Mont., where he lived until he was seventeen years of age and had completed high school. His mother died in his late boyhood and a loving aunt, Margaret C. Gillette, gave him a mother's care and in return received his sincere and constant affection.

Mr. Fraser's schooling was practically completed at the outbreak of the World War in 1917, and the University of Washington at Seattle granted him a Bachelor of Science in Civil Engineering degree early in that year in order to permit him to prepare to enter the Army. His service record as prepared by the War Department the day following his death may well be quoted, as follows:

"Garrett Alexander Fraser was admitted to the first Reserve Officers' Training Camp, Vancouver Barracks, Washington, May 15, 1917, organized under the provisions of Section 54, National Defense Act, where he trained, as a civilian candidate for a commission, until the close of the camp August 14, 1917, when honorably discharged.

"He enlisted on August 15, 1917, at Vancouver Barracks, Washington, as a private in the Engineers; was assigned serial number 2 439 404, and joined Headquarters, 316th Engineers, 91st Division, at Camp Lewis, Washington. He was appointed Battalion Sergeant-Major, September 13, 1917. He was in training at the Engineers Training Camp, Camp Lee, Virginia, December 28,

* Memoir prepared by Roy E. Miller, M. Am. Soc. C. E.

1917, to March 17, 1918, for a commission and was commissioned a Second Lieutenant, Engineers, March 18, 1918. He continued in training at Camp Lee, Virginia, until transferred in April, 1918, to Camp Dodge, Iowa, where he joined Company B, 528th Engineers, Service Battalion; sailed with it from the United States July 9, 1918, and served with this regiment in France, participating in the St. Mihiel offensive, September 12-16, 1918. He was promoted to First Lieutenant, Engineers, March 26, 1919. He returned to the United States May 30, 1919, and was honorably discharged at Hoboken, New Jersey, June 6, 1919, by reason of the demobilization of the emergency forces."

On his return home after demobilization Mr. Fraser attended the Butte School of Mines for a few months, later actually working in the Butte mines in the vicinity of the School. In January, 1921, and for the next eight years, he served on the staff of the Puget Sound Bridge and Dredging Company of Seattle, becoming a Vice-President in 1928. In these years, he was instrumental in bringing to a successful conclusion many engineering works of a general nature such as bridges, dredging, subaqueous tunnel, and a large multiple-arch dam, located in different parts of the United States, Canada, and Mexico. At the time of his death he had but recently moved to New York City and had become Assistant Secretary and Treasurer of the Metals Mining Shares, Incorporated, and of the Minerals Research Corporation. He is known best for having built the International Bridge across the Rio Grande between Brownsville, Tex., and Matamoras, Tamaulipas, Mexico, which was completed and opened for traffic for the Gateway Bridge Company on July 4, 1928.

Mr. Fraser passed through his thirty-four years of life with all the loyalty, courage, and zest, and with all the joy and laughter that can come to healthy, clean-minded men of his day and generation, which is to say that none was more blessed. He was saved from bodily and mental injury in two years of service in France during the World War, only to be brought down to a death as sudden and dramatic as war itself: and yet it was a death that one who knew him intimately realized he would accept as wholly fitting to his life and thoughts—a death by an instrumentality of this new world in which he lived.

Mr. Fraser was killed when the airplane in which he was a passenger crashed in taking off at Tampico, Tamps., Mexico, on April 10, 1929. He was en route from the City of Mexico to Brownsville, Tex., and was one of five in the plane, all of whom were killed, so that none was left to tell the true cause of the crash. It is said, however, that planes were changed at Tampico, the change being hurriedly made and insufficient time being taken to properly warm up the engine, so that after making a height of perhaps 300 ft., the engine failed. Thereupon the pilot attempted to volplane back to the aviation field and in banking for the turn without power lost complete control of the plane. As in all other new lines of human endeavor, sacrifices seem necessary as a corollary to the advancement of the art of flying. It is difficult to take a philosophical point of view of this cost in human lives to accomplish progress in the development of the world's new machines when one loses a companion and a devoted friend.

Of the things that Mr. Fraser accomplished, none was so outstanding as the promise of future deeds. His life had been so largely one of preparation—his talents so versatile. His day had but just arrived.

His dominant traits were loyalty to his associates and friends; an abiding interest in all who came within his circle of acquaintanceship; and the ability to make himself understood and sincerely liked by every one. His mind was active and quick, and he had a full measure of initiative and energy. Because of natural modesty and because his main interests were in another direction, only a few of his companions knew that Mr. Fraser had rare mathematical ability which amounted to almost that of a genius. He belonged in that branch of the Engineering Profession from which business executives are raised.

Mr. Fraser was elected an Associate Member of the American Society of Civil Engineers on March 15, 1926.

CLIFFORD LYNDE, Assoc. M. Am. Soc. C. E.*

Died May 21, 1929.

Clifford Lynde, the son of James and Lizzie E. (Clifford) Lynde, was born at Chelsea, Mass., on November 16, 1884.

After graduating from the Massachusetts Institute of Technology in 1906, Mr. Lynde was located in Oil City, Pa., for a year. From there he went to the New York Board of Water Supply as Assistant Engineer, with headquarters in Walden, N. Y. He was employed on the construction of the Catskill Aqueduct until 1914, when he moved to Brooklyn, N. Y., where he was engaged on subway construction by the Cranford Company. At the completion of this work, he was employed for a short time by Prentice Sanger, Architect, returning to Walden in 1918 to make special studies for proposed new equipment for the Walden Knife Company.

During this period, Mr. Lynde suffered an attack of influenza and, later, rheumatic fever with resultant heart trouble, from which he never completely recovered. For many months he made a brave struggle to regain his health and, in an effort to obtain strength, he moved to Miami, Fla., where he resided from 1923 to 1928. He established himself in business as a Manufacturer's Agent and was very successful in this venture, surviving the boom and deflation times as well as the great hurricane. Because of failing health, he again returned north to spend six months in a Montclair, N. J., Sanitarium. He survived until May 21, 1929, when he passed away in Walden, N. Y.

Mr. Lynde was married to Mildred Fowler, in Walden, on June 11, 1912. He is survived by his widow, his mother, and two children, Elizabeth A. and Fairfield F. Lynde.

"Cliff" will long be remembered by his many friends for his genial and happy disposition, his unfailing optimism, and his ability to surmont cheerfully for years the greatest of obstacles—ill health. In his passing, those who counted themselves among his friends have sustained a real and genuine loss.

Mr. Lynde was elected a Junior of the American Society of Civil Engineers on December 1, 1908, and an Associate Member on November 28, 1916.

* Memoir prepared by Dean G. Edwards, M. Am. Soc. C. E.

JAMES JOSEPH WALL, JR., Assoc. M. Am. Soc. C. E.*

Died August 16, 1929.

James Joseph Wall, Jr., the son of James Joseph Wall and Amelia Mary (Jones) Wall, was born on February 24, 1892, at Saginaw, Mich.

After graduating from the Central High School at Duluth, Minn., Mr. Wall entered Cornell University and was graduated from the College of Civil Engineering in 1916. During vacations previous to his graduation, he served as Chainman and Rodman for the Canadian Northern Railways and as Concrete Road Inspector at Superior, Wis., for the Portland Cement Association of Philadelphia, Pa. While at Cornell, Mr. Wall was active as a member of the Sigma Nu Fraternity, Manager of Fencing, and Advertising Manager of the "Cornell Civil Engineer".

In April, 1917, Mr. Wall entered the First Officers' Training Camp, at Plattsburg, N. Y., and served during the World War as First Lieutenant, Officers' Reserve Corps, 6th U. S. Engineers, and as a Captain of Engineers, U. S. Army, in command of A Company, 2d Engineers, Second Division, American Expeditionary Forces. He was cited by General Pershing and Major General Lejeune, of the U. S. Marine Corps, and also by General Buat, of the French Army, for bravery and gallantry, particularly in the rapid launching (in 7 min.) of bridges, under heavy fire, by his Company over the Meuse River during the night of November 10, 1918. These footbridges were completed ahead of schedule and resulted in the elimination of enemy machine gun nests with small losses for the Allies.

After his discharge from the Army, Mr. Wall was employed, successively, in the Sales Department of the Lakewood Engineering Company, Cleveland, Ohio, and as Engineer in charge of road grading on Route No. 3, Whitehorse Pike, New Jersey. From February to June, 1920, he was Assistant Field Engineer on Irrigation Construction for the Barahona Sugar Company, Santo Domingo, and on highway location and construction for the Dominican Republic. Following employment on valuation work by the Savage Arms Plant, Utica, N. Y., and as Engineer on Concrete Road Construction at Freehold and Spring Lake, N. J., in 1921, Mr. Wall became Sales Representative for the Holt Manufacturing Company, which position he held until September, 1923. After being connected with the Portland Cement Association, Pittsburgh, Pa., in concrete products promotion in 1924, he became Sales Representative for the Crescent Portland Cement Company, with headquarters at Dayton, Ohio, which position he held until shortly before his death.

His health failing, Mr. Wall went to Texas, where he was engaged for a short time on work for the Atchison, Topeka and Santa Fé Railway Company at San Angelo, Tex. He was obliged, however, to give up his work and enter the United States Veterans Hospital at Legion, Tex., where he died of tuberculosis on August 16, 1929.

* Memoir prepared by William L. Havens, M. Am. Soc. C. E.

He had an exceptionally pleasing personality which not only made him popular among his fellow students during his college days, but also served him well in his later employment in sales work. His success as an engineer salesman was no less marked than his bravery as a soldier.

Mr. Wall was elected an Associate Member of the American Society of Civil Engineers on December 14, 1925.

JOSEPH LLOYD CAGNANI, Jun. Am. Soc. C. E.*

Died August 2, 1929.

Joseph Lloyd Cagnani, the second son of Enrico and Palmira Zavatoni Cagnani, was born in New York, N. Y., on July 15, 1904. He was educated in that city and was graduated in 1928 from Cooper Union, with the degree of Bachelor of Science in Civil Engineering.

It was during this schooling period in 1922, which consisted mainly of evening-session study, that Mr. Cagnani entered the service of the Cutler-Hammer Manufacturing Company of New York, in the capacity of Draftsman on conveyor layouts. Later, in 1923, he became Estimator, Draftsman, and Designer for the Harris Structural Steel Company. His unusual ability and persevering efforts were quickly recognized and won for him the appointment of Assistant to the Chief Engineer.

Mr. Cagnani was intensely interested in mechanics, and, with the development of the airplane and aeronautics, he was eager to enter that field. Imbued with this one idea, he matriculated for post-graduate work in aeronautics at the Guggenheim School at New York University. His inclination and his conscientious application to this branch made him an excellent student, and, convinced of his physical and mental eligibility, he decided to enter the field.

In October, 1928, he became a commercial pilot for the Barrett Airways, Incorporated, at Armonk, N. Y. As he possessed a pedagogical turn of mind and an entertaining personality, and was also equipped with concrete facts and principles, he succeeded, in the spring of 1929, to the position of Director of the Ground School—a position that he filled with remarkable aptitude and understanding.

Subsequently, on August 2, 1929, Mr. Cagnani passed the examination for Transport Flier, attaining the highest average ever given for that test. Rejoicing in his new achievement, later on the same day he was flying a "joy plane" with two young friends, when motor and wing defects and disabilities caused him to fall 2 500 ft. It later appeared that he had fought death calmly and coolly every foot of the way down, as the inspectors found him with one hand on the ignition switch, to prevent fire, and the other on the controls.

He died as he had lived, a gentleman unafraid. "Big Joe", they called him, and he was big in heart and soul as well as in body. Four qualities go

* Memoir prepared by Mrs. Arthur Levin, Lazar Levin, and Charles W. E. Schroeder, Jun. Am. Soc. C. E.

to make a man—courage, strength, loyalty, and intelligence—and all four Mr. Cagnani had in abundance. His death was a loss not only to his friends, but to all who knew him.

Mr. Cagnani was elected a Junior of the American Society of Civil Engineers on October 1, 1927.

LEWIS IRVING FLETCHER, Affiliate, Am. Soc. C. E.*

Died August 12, 1929.

Lewis Irving Fletcher, the son of Lewis E. and Lucy Ellen (McCracken) Fletcher, was born in Marlborough, Mass., on December 7, 1865. He was educated in the schools of his native town and, later, attended a business college in Boston, Mass.

At the beginning of his professional career, Mr. Fletcher was engaged in construction for the New England Telephone and Telegraph Company, after which he built and operated the lighting plant for the Nashua, N. H., Electric Light Company. In 1889, he again built and operated an electric light and power plant for the Lowell, Mass., Electric Light Corporation.

For a time thereafter, he was engaged in the machinery business in Boston, subsequent to which he went to Easton, Pa., where as Chief Engineer he constructed a hydro-electric plant on the Delaware River for the Easton and Lehigh Power Companies. In 1902 and 1903 he was located at Bulls Bridge, Conn., in connection with the power development on the Housatonic River at that place, and in 1904 he served as Consulting Engineer for the Town of Easton. From 1905 to 1907 he was engaged as Engineer on Dam Construction for the American Pipe and Construction Company of Philadelphia, Pa. During this period he built twenty-two reservoirs and dams along the main line of the Pennsylvania Railroad, in Pennsylvania.

During 1909 and 1910, Mr. Fletcher was employed with the Ambursen Company at Rapidan, Minn., on the construction of a dam and power plant, and with H. M. Byllesby and Company, of Chicago, Ill. Later, he went to Estacada, Ore., where he finished the construction of a dam and hydro-electric plant on the Clackamas River. In 1912, he completed dams and paper mills for the Cornell Wood Products Company, in Cornell, Wis.

He went to Kent, Ohio, in 1913, where he was engaged in general contracting business, building roads and bridges in Northeastern Ohio. From 1921 to 1924, he served as Hydraulic Engineer and Resident Manager for J. Livingston and Company, Inc., of New York, N. Y., at Judsonia, Ark. In 1926 he was engaged on work for the United Gas Improvement Company, at New Milford, Conn., and at the time of his death, he was with the Davey Compressor Company, of Kent, Ohio.

Mr. Fletcher's sudden death at his home in Kent was a shock to the entire community. He became ill as he was preparing to retire for the night and

* Memoir compiled from information on file at Society Headquarters.

died in less than an hour from an internal hemorrhage. His was a kind and lovable personality, a man who held the marked esteem of scores of friends and associates.

Mr. Fletcher was a member of the Masonic Blue Lodge, at Nashua, N. H.; of the Knights Templars and Consistory, at Lowell, Mass.; of the Shrine, at Boston, and of the Order of the Eastern Star and the Benevolent and Protective Order of Elks, at Kent. He was an attendant at the Christian Science Church.

He was married at Lowell, Mass., on October 14, 1890, to Annie A. Shedd, who, with their daughter, Mona, survives him. He is also survived by four brothers, Howard, of Detroit, Mich., Fred, of Marlborough, Mass., Percy, of Fairhaven, Mass., and George, of Newark, N. J.

Mr. Fletcher was elected an Affiliate of the American Society of Civil Engineers on February 6, 1912.

JOHN ROBERT STANTON, F. Am. Soc. C. E.*

DIED APRIL 28, 1927.

John Robert Stanton, the son of John and Elizabeth R. Stanton, was born in New York, N. Y., on September 25, 1857. He was educated in the public schools of New York and was graduated from the School of Mines at Columbia University.

Mr. Stanton's career as a Mining Engineer was begun in 1879, in connection with the Atlantic Mining Company and the Central Mining Company of Michigan. In 1890, he became Secretary-Treasurer and Director of the Wolverine Copper Mining Company, and eight years later he was elected Treasurer of the Mohawk Mining Company, of which Company he afterward became President. He also acted as President and Director of the Fort Mountain Talc Company and the White Pine Extension Copper Company of Michigan. His further associations in the field of copper mining were with the Winona Copper Company and the Michigan Copper Company, during which period he was engaged in the active management of eight Lake Superior mines.

Having come from a family of noted metallurgists, Mr. Stanton was especially well posted on the Lake Superior Copper Region. His father had been rated as one of the best informed men on copper in America, and it was estimated in 1905 that the output of the copper mines in which he was interested reached 90 000 000 lb. per year. This fact may account somewhat for Mr. Stanton's interest and accurate knowledge of the business in which he remained active until 1918 when he retired on account of poor health. He left New York to live at his country home at Galesville, Wis., where he died April 28, 1927.

From 1876 to 1887, he was a member of the old Seventh Regiment, N. G. N. Y., later serving six years as Lieutenant and four years as Captain. He was a life member of Company A, Seventh Regiment Veterans Association.

* Memoir prepared from information on file at Society Headquarters.

Mr. Stanton was a member of the following societies: The American Institute of Electrical Engineers; the Lake Superior Mining Institution; the Franklin Institute of Philadelphia; the American Association for the Advancement of Science; the National Geographic Society of America; the American Forestry Association; the New York Botanical Gardens Society; the New York Zoological Society; the Horticultural Society; St. George's, St. Andrew's, and the Robert Burns Societies; the Huguenot Society; Sons of the Revolution; the Municipal Art Society; the Thomas Hunter Association; the Society for the Prevention of Cruelty to Children; and the Society for the Prevention of Cruelty to Animals.

He was also active in the following clubs: The New York Yacht Club; the Columbia Yacht Club; the Onigoaming Yacht Club; the Union League Club; the Lotus Club; the Engineers Club; the Republican Club; the Twilight and Dunwoody Country Club; the Chicago Athletic Club; and the Niscowabic Club.

Mr. Stanton was married on September 4, 1899, to Helen Maud Kilmer who, with a sister, Mrs. J. W. Moore, of Haughton, Mich., survives him.

Mr. Stanton was elected a Fellow of the American Society of Civil Engineers on April 4, 1899.